NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
REPORT **90**

PROTECTION OF STEEL IN PRESTRESSED CONCRETE BRIDGES

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PROTECTION OF STEEL IN PRESTRESSED CONCRETE BRIDGES

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RESEARCH SPONSORED BY THE AMERICAN ASSOCIATION
OF STATE HIGHWAY OFFICIALS IN COOPERATION
WITH THE BUREAU OF PUBLIC ROADS

SUBJECT CLASSIFICATION:
BRIDGE DESIGN
CEMENT AND CONCRETE
CONSTRUCTION
GENERAL MATERIALS
MAINTENANCE, GENERAL

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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In recognition of these needs, the highway administrators of the American Association of State Highway Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Bureau of Public Roads, United States Department of Transportation.

The Highway Research Board of the National Academy of Sciences-National Research Council was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as: it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state, and local governmental agencies, universities, and industry; its relationship to its parent organization, the National Academy of Sciences, a private, nonprofit institution, is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

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The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

NCHRP Report 90

Project 12-5 FY '67
SBN 309-01877-3
L. C. Card No. 74-606084

Price: $4.00

This report is one of a series of reports issued from a continuing research program conducted under a three-way agreement entered into in June 1962 by and among the National Academy of Sciences-National Research Council, the American Association of State Highway Officials, and the U. S. Bureau of Public Roads. Individual fiscal agreements are executed annually by the Academy-Research Council, the Bureau of Public Roads, and participating state highway departments, members of the American Association of State Highway Officials.

This report was prepared by the contracting research agency. It has been reviewed by the appropriate Advisory Panel for clarity, documentation, and fulfillment of the contract. It has been accepted by the Highway Research Board and published in the interest of an effectual dissemination of findings and their application in the formulation of policies, procedures, and practice on the subject problem area.

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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FOREWORD

By Staff

Highway Research Board

This report should be of special interest to design engineers, construction engineers, prestressed concrete manufacturers, and others concerned with the use of prestressed concrete in locations exposed to potentially corrosive conditions. In addition, specification writers and members of committees concerned with updating specifications for prestressed concrete should find the contents valuable. The report contains specification recommendations intended to encourage a uniformly high quality of prestressed concrete. Based on the findings from a comprehensive literature review, a survey of field experience, and various laboratory and field tests, it is believed that adoption of these more stringent specifications will be a step forward.

Increased use of prestressed concrete utilizing higher strength tendons, longer strands, and decreased spread between working and ultimate stress has focused concern on the service life of the tendons. Experience has shown that under certain severe corrosive conditions prestressed concrete members may suffer damage and short life. This in turn has raised questions as to methods of protecting the tendons from corrosion both before and after encasing and grouting. As a first step, current grouting systems needed to be evaluated as to their adequacy. Also, research was necessary to determine environmental conditions under which special protection is required and to develop effective protective systems.

The Denver Research Institute used a many-faceted approach to investigate this problem during the two years of work under NCHRP Project 12-5, "Protection of Steel in Prestressed Concrete Bridges." The Institute first undertook an extensive literature search to ascertain the state of the art. Next, a survey of many agencies was performed to determine whether field problems with prestressed concrete members did in fact exist to a significant degree. The survey pinpointed certain locations of alleged distressed prestressed concrete members that were later subjected to detailed study by the investigators. Laboratory work dealing with accelerated corrosion testing, protective coating testing, grouting studies, and other relevant evaluations proceeded concurrently.

The findings in this report present information of value to those concerned with the protection of steel in prestressed concrete members. The conclusion regarding the quality of current practice will certainly be of interest to the engineers and manufacturers working with prestressed concrete. Although the conclusions regarding current practice are reassuring, recommended changes in the current specifications covering the materials and construction of prestressed concrete members would encourage uniform high standards throughout the industry. Included in this report are many interesting suggestions concerning research topics for further investigation in this problem area.

It must be stressed that inasmuch as bridges and other structures are designed for very long service lives, sometimes 75 to 100 years, a two-year study is less than absolutely conclusive in guaranteeing corrosion protection effectiveness over the design life. Certainly a continuing program of field inspection and evaluation of prestressed concrete members is necessary.

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ACKNOWLEDGMENTS

The research reported herein was performed under NCHRP Project 12-5 by the Denver Research Institute of the University of Denver. Jerry D. Plunkett, Associate Professor of Metallurgy, University of Denver, served as project director for the contract from September 1966 until his resignation from the University in March 1967. These duties were then assumed by William C. Hagel, Chairman of the Department of Metallurgy, who served in this capacity until the completion of the contract.

Completion of the study and preparation of the report were carried out by D. G. Moore, Senior Scientist, and D. T. Klodt, Research Metallurgist, both of the Institute; and R. J. Hensen, Assistant Professor of Civil Engineering at the University.

Grateful acknowledgment is made for the help of the following individuals, who assisted not only in the laboratory work but also in the reduction of data: George Crater, Ted Espinoza, and Milton King of the Metallurgy Division; and Henry Good, John Starek, and M. Marquez of the Mechanics Division.

Dr. Wen H. Huang, Assistant Professor of Civil Engineering, performed the theoretical analysis of the flow behavior of grouts given in Appendix G and assisted in several other phases of the grouting program.

Special acknowledgment is also due Professor R. Szilard who made important contributions to the project during the first year of its operation. Professor Szilard was not only a key figure in the early planning of the program and in the literature review, but he also conducted the survey of current prestressing practices described in Appendix J. After Professor Szilard resigned from the University in August 1967, the responsibilities for the grouting aspects of the investigation were assumed by Professor Allen G. Thurman, Acting Dean of Engineering at the University of Denver, who planned and supervised the grouting program as described in Appendix G.

The gratitude of the authors also is extended for the excellent cooperation provided by the bridge departments of all 50 state highway departments, as well as by Prestressed Concrete Institute, Portland Cement Association, CF&I Steel Corp., Armco Corp., Ideal Cement Company, Prestressed Concrete Company of Colorado, Prescon Corp., American Stress Wire Corp., and Boyles Galvanizing Company.

PROTECTION OF STEEL IN PRESTRESSED CONCRETE BRIDGES

SUMMARY

Studies were made of factors affecting the corrosion of steel in prestressed concrete bridges and of possible methods of minimizing the corrosion dangers. Both laboratory and field testing were employed.

Laboratory tests were conducted to determine if brittle fracture of prestressing steel could be induced by hydrogen embrittlement, stress corrosion, or H_2S cracking. The tests showed that neither hydrogen embrittlement cracking nor stress corrosion cracking occur when the prestressing steel is in a chloride-bearing concrete environment. However, H_2S cracking can occur and is considered to be a potentially serious problem if concrete cover is inadequate or if the pH of the concrete is lowered by carbonation.

Studies of corrosion mechanisms demonstrated that an oxygen-concentration cell is necessary before steel will corrode in a simulated concrete environment. Anodic polarization curves determined potentiostatically showed that chlorides in concrete destroy the passivity of the steel and that the threshold level for chloride below which corrosion will not occur is extremely low. Other studies demonstrated that large amounts of inhibitors, such as $NaNO_2$, are required to prevent corrosion of steel in simulated concrete environments containing chlorides.

Various types of protective coatings were applied to prestressing steel specimens and numerous tests were then made to evaluate performance. Coatings showing the greatest promise were hot-dipped galvanized and epoxy. However, neither met all of the requirements for an ideal coating. Galvanizing lowered steel strength and increased relaxation. Also, the zinc layer was not entirely inert in chloride-bearing concrete environments. Epoxy coatings provided excellent corrosion protection but were lacking in wear resistance and ease of application as well as in cost.

Tests with pretensioned beams exposed to 3.5% NaCl solutions showed that concrete covers of greater than ¾ in. are required to provide protection for the steel, even in short-time testing. Open cracks that were only 0.004 in. wide at the concrete surface permitted rapid corrosion to occur. Microcracks, on the other hand, were not harmful under the test conditions that were used.

Laboratory studies of the flow behavior of grouts provided information on such variables as water-cement ratio, type and amount of additive, and mixing times. Full-scale grouting tests were made using the findings of the laboratory experiments. Ducts with a maximum length of 321 ft and a steel-to-duct area ratio of 0.25 were grouted successfully. A theoretical expression was derived for computing pressure drops, flow rates, and maximum pumping distance for grouts pumped into ducts with varying steel-to-duct area ratios.

Examination of steel from fragments of prestressed bridges and piles that had been exposed in severe environments for extended periods showed virtually no corrosion or loss in properties of the steel. These observations combined with the many other findings obtained during the course of the study resulted in the conclusion that concrete when properly used can provide satisfactory long-time corrosion protection for prestressing steel.

CHAPTER ONE

INTRODUCTION AND RESEARCH APPROACH

The first prestressed concrete bridges were installed in the United States in the early 1950's. In the years that followed the use of bridges of this type grew rapidly and there are now more than 14,000 such structures in the nation's highway system.

In the great majority of cases, these bridges have given trouble-free service with little if any evidence of corrosion of the tendons. However, even the oldest bridges are less than 20 years old. Also, the present trend is toward the use of higher-strength tendons, longer spans, and a decreased spread between working and ultimate strength. All of these factors are undoubtedly responsible for a growing concern among engineers about possible corrosion damage to tendons, especially after long-time service. This concern exists even though concrete is known to provide an almost ideal environment for the protection of steel under most exposure conditions. The protective action stems not only from the high alkalinity of the concrete but also from its relatively low permeability to water and oxygen. When chlorides are present, however, such as from deicing salts or from sea spray, the protection tends to break down and in this type of environment corrosion of reinforcing steel has been known to occur. Also, the possibility of brittle fracture of prestressing tendons from either stress corrosion or hydrogen embrittlement has been advanced by a number of corrosion specialists. Failures of this type can occur with no macroscopically observable plastic deformation of the steel and with no prior warning; hence, brittle fractures of tendons could be especially dangerous from the standpoint of bridge safety.

The goals of the investigation described in this report were two-fold. The first was to determine the environmental conditions under which special protection of prestressing steel was required, and the second was to develop effective protective systems for both pretensioned and post-tensioned girders for those environments in which additional protection of the steel was indicated. In pursuing these goals the investigation was divided into a number of phases, consisting of:

1. A survey of domestic and foreign data on corrosion and prevention of corrosion of prestressing steel in bridges and other structures, including circulation of a questionnaire to determine the experience of bridge engineers with the corrosion of bridge tendons.
2. A review of practices in current use for preventing corrosion of tendons.
3. Field inspections of bridges and related structures to ascertain the extent of any damage to tendons.
4. Experiments to identify more clearly the corrosion mechanisms as an aid in determining the direction needed in the development of protective systems.
5. Selection and testing of protective coatings.
6. Tests to determine the effectiveness of concrete cover in preventing steel corrosion.
7. Laboratory and field testing of the effectiveness of grouting methods.
8. Preparation, where applicable, of recommended specifications for (1) protective systems, (2) handling, shipping, and storing of tendons, (3) methods of prestressing, (4) methods of grouting, and (5) thickness of concrete cover.

The investigation was concerned for the most part with bridge superstructures. However, many of the findings apply almost equally well to other types of prestressed structures, including prestressed piles. Also, although the research was directed, in part, to post-tensioned bridge superstructures, no attention was given to the protection of ungrouted tendons inasmuch as most bridge specifications in the United States require that the tendons be grouted.

Finally, it should be pointed out that highway bridges are expected to have a useful service life of 75 years or more. Obviously, a study conducted over only a 2-year period could not be expected to provide complete reliability with respect to corrosion damage of the prestressing steel that might occur after 75 or 100 years. The right kind of research, however, could give good indications of long-time damage and every effort was made to plan and conduct the present research on this basis.

CHAPTER TWO

FINDINGS

Perhaps the most significant result of the 2-year study was the finding that steel corrosion is not now a serious problem with prestressed concrete bridges, and that with proper precautions it need not be a problem in the future. Of the many thousands of prestressed concrete bridges now in service in the United States, only a limited few (less than 0.007%) have shown evidence of steel corrosion (Appendix A). Furthermore, up to the present there has not been a single case of a catastrophic failure of a prestressed bridge reported anywhere in the world (Appendix B). In rare occasions, fractures of wires and strands have been reported, but these fractures have almost always occurred either during or shortly after prestressing. Also, with proper attention to design and construction practices, even these fractures could have been avoided.

Inspection of steel taken from pretensioned concrete fragments—after exposure for prolonged periods to environments many times more severe than any that would be encountered in a bridge superstructure—showed a virtual absence of any corrosion damage (Appendix H). In fact, the only evidence of corrosive attack was at extreme ends of the fragments where corrosion had penetrated along the strands for a short distance.

Penetration of corrosive solutions along the interstices surrounding the center wire in seven-wire strand (center-wire corrosion) was found not to be a problem with pretensioned concrete. In all samples that were examined, cement paste had penetrated into the interstices during vibratory compaction of the concrete (Appendices F and H). The presence of the hardened cement paste prevented the free capillary flow of corrosive solutions along the center wire.

BRITTLE FRACTURES OF TENDONS

Tests of the type described in Appendix C showed that stress-corrosion cracking of prestressing steel does not occur in chloride-containing concrete environments, even under highly accelerated test conditions. Brittle fracture of the steel, however, will occur sometimes in a matter of minutes in the presence of hydrogen sulfide. The time for failure increased with decreasing H_2S content. Also, when the steel was protected with a cover of cement mortar, 0.25 to 1.25 in. in thickness, no failures occurred during the maximum testing time of 10 hr. Galvanizing protected against H_2S cracking, but only if the zinc layer on the steel was continuous.

Brittle fractures from hydrogen embrittlement were not observed under test conditions simulating those that have been reported to cause hydrogen embrittlement failures in the field. Sulfides present in cement or aggregate could, however, be a contributing factor to failures reported in the field.

CORROSION CELLS

Studies described in Appendix D with various corrosion cells similar to those that might occur in chloride-containing concretes indicated that oxygen-gradient cells were potentially the most serious. Corrosion of the prestressing steel did not occur in these cells unless a difference in oxygen concentration was present. Also, even with oxygen gradients, no corrosion occurred unless chlorides were present in the environment.

Polarization measurements with a potentiostat demonstrated that even minor concentrations of chloride in simulated concrete environments destroyed the passivity of the steel. Other studies demonstrated that relatively large amounts of inhibitors, such as $NaNO_2$, are required to prevent corrosion of prestressing steel in simulated concrete environments containing chlorides.

PROTECTIVE COATINGS

The two coating types (Appendix E) showing the greatest promise for the protection of prestressing steel were the hot-dipped galvanized and the epoxies. Neither type, however, met all requirements for an ideal coating. Galvanized coatings lowered steel strength and increased relaxation. Also, the zinc layer was not entirely inert in concrete that contained chlorides. Hydrogen embrittlement by cathodic charging of the steel did not occur for stressed specimens with breaks in the galvanized layer that were tested for long times in a chloride environment.

Epoxy coatings provided excellent corrosion protection but were lacking in wear resistance and ease of application. They were also relatively high in cost.

Two systems were found to be suitable for protecting prestressing steel during shipping and storing. One was the currently used vapor phase inhibitor in which crystals of the material are placed together with the steel in sealed enclosures; the second was a sodium silicate-sodium nitrite film applied to the steel as a rust-inhibiting film. Neither of these protective systems had any important deteriorating effect on bond development between the steel and concrete. A water-soluble oil film also provided good corrosion protection. However, it could not be removed by flushing with water, and the remaining oil film caused an excessive loss in bond development.

Protection of prestressing steel by application of a sealant-type coating to the concrete surfaces of a girder was not investigated. However, an analysis of this approach (Appendix B) indicated that it was not a promising method for protecting the steel.

CONCRETE COVER

As indicated in Appendix F, tests with pretensioned beams in a controlled environment showed that a 0.75-in. concrete cover was insufficient when chlorides were present. No steel corrosion was observed under the test conditions when the concrete covers were 1.5 in. or greater.

Improved compaction with fewer voids between the steel and concrete was observed with the thicker concrete covers. Cracks in the concrete as narrow as 0.004 in. at the outer concrete surface permitted rapid steel corrosion to occur. Microcracks, on the other hand, were not detrimental.

In those cases where corrosion occurred, the corrosion tended to concentrate at voids between the steel and concrete. The protectiveness of the concrete cover did not appear to be affected by loading levels as long as the levels were maintained below those required to cause open cracks to appear in the concrete cover.

GROUTING

Laboratory tests with grouts indicated that a mixture with a water-cement ratio of 4.5 to 5.0 gal per sack of cement provided the most effective mix. The use of a proprietary additive in the grout significantly increased fluidity, with an accompanying reduction in the required pumping pressure. At the same time, the additives decreased the 28-day compressive strength of the grout by as much as 60%.

Field trials of the grouting in ducts up to 321 ft long showed that grouting could be achieved at reasonable pressures. Voids that were present in the cured grout were of the order of ⅛ in. in diameter. An exception to this occurred along the top surface of the ducts, where free water collected and migrated to the low points in the drape of the tendons. This water, after absorption into the hardened grout, left an elongated void near the duct wall.

Laboratory experiments showed that unless the confining pressure of the grout is maintained during the early stages of curing, the grout gives up free water. Proprietary additives did not eliminate this water formation. In fact, laboratory tests showed evidence of increased water formation in slurries containing additives. Grouts with additives were observed to shrink for as long as 15 to 20 min before they began to expand. Maintaining the confining pressure after grouting was also found to minimize the size and number of internal voids caused by the presence of entrapped air.

The field test demonstrated that a 2-in. I.D. duct 321 ft long with a tendon-to-duct area ratio of 0.25 can be pumped at a grouting pressure of 80 psig. The field test also showed that the use of excessive pumping pressure will cause the water to separate from the grout, resulting in a "flash set" and stoppage of any further flow through the duct.

A theoretical expression was derived for computing pressures required for grouting ducts of various lengths and with various steel-to-duct area ratios. Fluidity parameters of the grout that are needed for the calculation can be obtained from measurements made with a viscometer. The expression was found to predict very closely the pressures that were observed during the grouting of actual ducts.

PROTECTION OF END ANCHORAGES

With properly grouted tendons, failure of end anchorages by corrosion is not dangerous to the structural integrity of a bridge. The tendons should be well bonded after a satisfactory grouting application, and for this reason no significant loss of prestress would occur in most ordinary installations, even though the anchorage failed by corrosion. Nevertheless, anchorage corrosion can cause unsightly staining of the concrete and, also, the continued integrity of anchorages would provide added insurance against loss of prestress in the event the grouting was not 100% effective. This insurance becomes especially important when large highly stressed tendons are confined in a relatively small cross-sectional area of a girder.

The findings of this study, based on an analysis of earlier work, are that epoxy concrete cappings provide the best corrosion protection for exterior anchorages. However, air-entrained portland cement concrete provides good protection if the anchorages are recessed. Any cracks that appear around either type of capping should always be caulked as soon as possible to prevent air and water from reaching the anchorage.

AASHO BRIDGE SPECIFICATIONS

The requirements of the AASHO bridge specifications for prestressed concrete should be tightened so as to provide improved insurance against possible future corrosion damage to tendons. In addition to specifying the minimum concrete cover, requirements are desirable on prestressing steel, on types of concrete, and on grouts and grouting procedures. Appendix K lists the new requirements that should be incorporated.

CHAPTER THREE

INTERPRETATION, APPRAISAL, AND APPLICATION

INTERPRETATION

Both the literature review and the work conducted with corrosion cells are clear in showing that prestressing steel embedded in sound concrete does not corrode when chlorides or other corrosive ions are absent from the environment. Therefore, an extremely long service life can be expected from the great majority of prestressed concrete bridges in the United States. These, of course, are the bridges erected in non-coastal areas where deicing salts are either not used, or, if they are, the design is such that there is little if any opportunity for the salt solutions to reach the girders. The findings of the present study indicate that corrosion-free service of 50 to 100 years is not only possible for such bridges, but also highly probable.

For bridges exposed in marine environments, or for those in which the girders are exposed to runoff water containing deicing salts, steel corrosion after long-time exposure is always a possibility. Even the best concrete is not completely impervious to chloride solutions and, given sufficient time, chloride ions will eventually reach the steel. However, the present study has shown that corrosion will not occur even under these conditions unless oxygen is available at the steel surface. In addition, voids, cracks, or local differences in concrete permeability to oxygen would also need to be present so as to generate differences in oxygen concentration at the steel. In high-quality prestressed concrete, inhomogeneities of this type should be minimal. Therefore, even in a chloride environment, long-time, corrosion-free service should be obtained from a prestressed concrete bridge. The excellent condition of the steel in the pretensioned pile fragment exposed for 14 years to twice-daily immersions of sea water (Appendix H) is convincing evidence that such long-time protection is possible.

The prediction of freedom from steel corrosion for extended time periods in both chloride-free and chloride-containing environments is based on the premise that the concrete cover over the steel does not deteriorate from some such cause as carbonation, spalling by freezing and thawing, alkali-aggregate reactions, or acid attack from an industrially polluted atmosphere. Also, it presupposes that the original concrete cover was adequate, and also that the concrete was well compacted and reasonably free of voids. However, because it is impossible to have 100% assurance that all of these conditions are met, periodic inspection of prestressed bridge structures for evidence of concrete deterioration and steel corrosion (Appendix I) becomes highly important. For most bridges, annual inspections should be sufficient.

From the results of the laboratory work, brittle failures of the prestressing steel in bridges from either stress corrosion or hydrogen embrittlement need not be a cause of great concern because the study showed that failures of this type are unlikely to occur in any normal bridge environment. However, if H_2S is present in the atmosphere, or if sulfides are present in the cement or aggregate, H_2S cracking of the steel cannot be ruled out as a possibility. Although, in the laboratory testing, brittle fractures did not occur when either zinc or aluminum was coupled with the steel in simulated corrosive environments, coupling of prestressing steel to any active metal, including zinc and aluminum, can be considered an undesirable practice from the standpoint of corrosion theory. The possibility always exists that, under certain corrosive conditions, hydrogen embrittlement failures could occur from cathodic charging of the steel.

With reference to the AASHO bridge specifications, some reasoning beyond that directly supported by the experimental evidence was used in arriving at the conclusion that the specification needs strengthening. The sole requirement on corrosion protection in the present specification is one on minimum concrete cover. To ensure the best long-term protection it seems important also to include requirements not only for the prestressing steel but also for the concrete and grout. Because the new requirements as suggested in Appendix K conform with currently used practices, no increase in costs of prestressed members would be anticipated if the additional requirements were incorporated.

APPRAISAL

Complete reliability with respect to the kind of corrosion that might occur in 75 to 100 years is difficult to achieve in a study conducted over only a 2-year period. The varied research approach taken in the investigation should give good indications of long-term durability; however, it is obvious that research needs to continue if the best possible reliability for predicting long-term behavior is to be achieved. Chapter Four outlines the research that is still needed.

APPLICATION

Although the findings of the research (as given in Chapter Two and in the various appendices) will be of interest to all fabricators and users of prestressed concrete, the information will probably be of greatest value to committees responsible for preparing specifications for prestressed concrete bridges.

CHAPTER FOUR

CONCLUSIONS AND SUGGESTED RESEARCH

CONCLUSIONS

The following generalized conclusions appear to be justified as a result of the investigation:

1. Brittle fracture of prestressing steel by stress-corrosion cracking or by hydrogen embrittlement does not occur in simulated concrete environments, even when chloride ions are present. Hydrogen sulfide cracking of the prestressing steel, however, can occur when H_2S is present at the steel surface.
2. Sound crack-free concrete with the thicknesses of cover recommended in the present AASHO bridge specifications will provide adequate corrosion protection for the prestressing steel. Coating of tendons for added corrosion protection is an unnecessary precaution, except for those bridges that are exposed to an extremely severe environment (for example, a severely polluted atmosphere) where an added insurance against long-time corrosion damage to the steel is desirable.
3. Prestressing steel needs to be protected during shipping and storing to prevent corrosion damage. It is particularly important to protect post-tensioning tendons when the steel is placed in ducts weeks or months prior to grouting. Recommended protective systems include (1) vapor phase inhibitors, (2) sodium silicate films on the steel tendons, and (3) flushing the duct with a saturated solution of calcium hydroxide after the steel has been inserted.
4. Grouting of long ducts (300 ft and longer) can be accomplished at reasonable pumping pressures if control is maintained over both grout fluidity and steel-to-duct area ratio. Voids between the tendons and grout can be minimized (if not eliminated) by maintaining the pumping pressure on the grout for several hours after the grouting is completed.
5. Corrosion of steel under severely corrosive conditions tends to occur at voids between the tendons and the concrete or grout. Through use of practices recommended in this report, concrete compaction and grouting of ducts can be accomplished by procedures that will minimize the formation of these voids.

SUGGESTION FOR FURTHER RESEARCH

Research is still needed in the following areas:

Permeation Rates of Oxygen Through Concrete.—Corrosion of prestressing steel cannot occur in the absence of oxygen; yet, with the exception of water vapor, no investigations have been made of the relative resistance of various concretes to gas permeation. Because the presence of air, and/or oxygen, at the steel surface is a deciding factor as to whether the steel will corrode, it is recommended that air permeation rates through various concretes be investigated with the concrete in both the saturated and unsaturated condition.

Carbonation Rates of Concrete.—Because carbonation of the concrete covering the tendons will, in time, destroy the corrosion protection provided by the highly alkaline cement, more needs to be known about the rates at which cement carbonation proceeds, as well as the various factors that affect this rate.

Deterioration of Concrete in Industrially Polluted Atmospheres.—Air pollution is increasing in the United States, and in the entire world. This means an increase in such acid gases as SO_2 and NO_2 in the atmospheres to which highway bridges will be exposed. Because acid attack, like carbonation, will in time destroy the corrosion protection of the concrete cover, it is important to learn more about this type of attack. It is recommended that an investigation be conducted to determine which of the gases are the most dangerous, and the effect of their concentration in the atmosphere on the rate of deterioration of various types of concrete.

Compaction of Concrete.—Although much is already known about compaction procedures and practices that will best achieve maximum concrete density, additional investigations are needed to determine the optimum procedures for minimizing voids at the tendon-concrete interface.

Additional Studies of Calcium Chloroaluminate.—Further studies should be conducted on formation of calcium chloroaluminate in concretes exposed to chloride environments. In studies of this type particular attention should be given to the effect of the formation of this compound on concrete permeability, electrical resistivity, strength, and dimensional stability.

Hydrogen Embrittlement of Tendons in the Presence of Active Metal Couples.—No hydrogen embrittlement failures of highly stressed wires were obtained in the present research when prestressing steel was coupled to zinc or aluminum under accelerated test conditions. This finding implies that brittle fractures will not occur when, for example, steel tendons are used in conjunction with a galvanized steel duct. The observed absence of brittle fracture, however, applies only to the set of test conditions that was used. Further long-time studies should be conducted to be certain that such failures will not occur in actual chloride-bearing environments.

Pressurized Grout Reservoirs for Maintaining Grouting Pressures During Curing.—The finding that voids in a post-tensioning duct system can be largely eliminated by maintaining a pressure on the duct until the grout has acquired its initial set warrants additional research on possible methods of maintaining this pressure. It is recommended that a grout reservoir pressured with air and attached to the duct inlet be investigated. In addition, the possibility of pumping grout into the ducts by this air pressure approach should not be overlooked. The results of the present study (Appendix G) indicated that large air bubbles were not introduced into the grouts when the plexiglas models were pumped in this way. The economics that might result from air pressure pumping, together with the ease with which pressures could be maintained during the setting period of the grout, would appear to justify an investigation of this approach with a full-sized system.

Development of an Improved Field Test Viscometer.—It is recommended that a new commercial viscometer be developed as a substitute for the flow cone that is currently used for the field testing of grouts. The new instrument should provide a measure of shear strain for various shear stresses so as to better characterize the grout. At the same time it should be of simple construction and easy to operate. Above all, the readings should be of a type that would permit the inspector to know if the fluidity he observes was obtained by mixing the grout in strict accordance with specifications, or if, on the contrary, it was achieved by the simple expedient of adding excess water to the slurry.

Inspection Method for Grouted Tendons.—Although use of suitable grouts and grouting practices as recommended in this report will result in grouting applications that are free of large voids, there can never be 100% assurance that some deviations from the recommended procedures have not occurred at the job site. Therefore, a nondestructive inspection technique for grouted tendons is needed, especially where large and expensive structures are involved. In England, many post-tensioned girders are now being inspected by gamma-ray radiography using a Co^{60} source. Although this approach has been shown to be capable of detecting some large voids in ducts, there is no assurance, at present, that the method will detect each and every one. More work needs to be done with this inspection method, using, if possible, the services of an organization with a broad background in techniques of this general type. (Mr. L. G. Spencer of North American Inspection Services, Ltd., 8845 63rd Ave., Edmonton, Alberta, has expressed a strong interest in development of radiographic inspection methods for grouted tendons.)

Development of Bridge Deck Designs with Improved Drainage.—It is recommended that additional attention be given to the design of bridge decks that will provide 100% assurance that runoff water containing deicing salts will not reach the prestressed girders.

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APPENDIX A

SURVEY OF STATE HIGHWAY DEPARTMENTS ON CORROSION OF STEEL IN PRESTRESSED CONCRETE BRIDGES

It was recognized shortly after the initiation of the research program that very little was known with certainty about the extent of steel corrosion in existing prestressed bridges in the U.S. highway system. Also, information was not readily available on the number, type, and age of existing bridges, or on what kinds of deicing practices were used in the various states.

To obtain answers to these questions, a questionnaire was mailed in November 1966 to the chief bridge engineer in each of the 50 states, as well as Puerto Rico and the District of Columbia. Excellent cooperation was achieved, with all 52 survey forms being returned. A summary of the results of the survey follows.

AGE OF OLDEST BRIDGES

According to the survey returns, the first prestressed bridges in the United States were installed in 1951 by six different states (California, Kentucky, Massachusetts, Michigan, Pennsylvania, and Tennessee). This means that the age of the oldest bridges in the country at the time of the survey (1966) could not have been more than 15 to 16 years (Fig. A-1).

NUMBER OF PRESTRESSED BRIDGES

The number of prestressed bridges in service in 1966 in the various states is shown by the bar graph in Figure A-2. In some cases, these numbers represent estimates by the bridge engineer, but they are believed to be correct within 10%.

The total number of prestressed bridges in service was approximately 12,000. Pennsylvania and Texas had the most (1,800 and 1,500, respectively) and Kansas had the least (2). Two states (Alabama and Maine) had none. The 12,000 figure may be a low estimate, however, because many of the bridge engineers did not report on the number of prestressed bridges in the county road system.

According to the survey, the great majority of the bridges (92.3%) were constructed with pretensioned members. However, in California about half of the estimated 500 bridges in service in 1966 were post-tensioned and eight of these post-tensioned structures had ungrouted tendons. Approximately 150 bridges were reported to be combined pretensioned and post-tensioned structures.

Figure A-3 shows the number of post-tensioned bridges by states in 1966. Twelve states had no bridges of this type; an additional 11 had only one or two. Arkansas, New Jersey, Pennsylvania, and California had the most.

DEICING PRACTICES

Almost all states confronted with snow and ice problems used sodium chloride and calcium chloride, usually in combination with sand or cinders. Some magnesium chloride was used by one state (California) and the occasional use of urea was reported by the District of Columbia. Of the northern states, only Montana, New Jersey, and Oregon used no deicing salts; Vermont used salts only in the city of Brattleboro.

VISUAL EVIDENCE OF CORROSION

The presence of rust stains of a type that might indicate corrosion of strands was reported by only three states. In two of the states (California and Pennsylvania) only one bridge was involved. In the third (Maryland), rust-like streaking was reported along the bottom flanges of several 50- to 70-ft pretensioned girders.

Both Michigan and New York reported isolated cases of spalling of concrete over prestressing steel. In Michigan two cases were observed where concrete had spalled off the bottom of pretensioned box beams exposing ⅜-in.-diameter strands. In both cases, water was free to penetrate into girders through cracks in the deck slab. The spalling was not caused by strand corrosion; instead, it was due to freezing of water that had collected in the interior box-shaped cavities of the beams. No drainage holes had been provided in the original design. In New York, some spalling of concrete in bridge girders was reported. Information on both the extent and exact location of the spalling with respect to tendons was not included in the return. Also, in New York, several county bridges showed spalling, with rusting of prestressing strands at the spalls.

FAILURES

Only one serious case of corrosion in a prestressed bridge was reported. This was in the state of Washington. The failure occurred at the Hood Canal Bridge (at Port Gamble), which is a floating bridge over sea water (Puget Sound). The pontoons, which are pretensioned, are strengthened and aligned by 1,000-ft post-tensioning cables. The cables (which are uncovered) pass through the hulls of the pontoons.

The bridge, which is 6,500 ft long, was opened for traffic in 1961. About 1½ years later, wires began to fail in the post-tensioning bundles. As failures occurred, wires were repaired. Pitting-type corrosion was observed on failed wires; it was believed by some that failures were due to stress corrosion.

After about 4 years, a massive failure occurred that involved 3 wires of a 40-wire bundle. A tar-like coating initially applied to the tendons did not provide satisfactory protection. Condensation is present in this installation, and, because the coating was not smooth and uniform, it tended to trap moisture. Periodic application of a protective grease coating to tendons is now employed as a maintenance procedure to minimize danger of future failures.

An 18-ft tide exists at the bridge site, and the bridge rises and falls with the tide. It is at least possible that the resulting movement could impose fluctuating stresses in the tendons, thereby creating conditions conducive to corrosion fatigue.

At the time of the survey, no evidence of corrosion had been observed in the pretensioning strands used for fabricating the pontoons.

The only other reported failures of tendons in bridge girders were those caused by traffic accidents, which usually involved trucks. However, only four mechanical failures of this type were reported.

EXPECTED SERVICE LIFE

The great majority of the engineers who replied to the questionnaire expected the structural members of a prestressed concrete bridge to last for 50 years. Eleven expected a 100-year life and two expected a life of greater than 100 years; only two would be satisfied with a life of 35 years.

OUT-OF-SERVICE BRIDGES

Out-of-service bridges could be of considerable value in assessing the amount of corrosion of the prestressing steel that had occurred. However, only three bridges of this type were reported—one in Colorado, one in Georgia, and one in Vermont. The first two were out of service because of flood damage, and the third was out of service because of relocation of a highway.

No Prestressed Bridges:
Ala., Maine

Not Reported:
La.

<6 YRS	6-8 YRS.	8-10 YRS.	10-12 YRS.	12-14 YRS	14-16 YRS.
		Ariz. - 1957			
		Colo. - 1957	Ark. - 1956		
		Kan. - 1957	Fla. - 1955		
		Minn. - 1958	Ga. - 1956		
		Miss. - 1957	Ida. - 1956	Alaska - 1954	
		Mont. - 1957	Nebr. - 1956	Conn. - 1953	Calif. - 1951
		Nev. - 1958	N.J. - 1956	Dela. - 1953	Ky. - 1951
	Hawaii - 1959	N. Car. - 1957	N.M - 1956	D C. - 1953	Mass. - 1951
	N. Hamp - 1960	N. Dak. - 1957	S Car - 1956	Ill - 1954	Mich. - 1951
	Okla. - 1959	N.Y - 1957	Texas - 1956	Ind. - 1954	Mo. - 1952
	S. Dak. - 1959	P. Rico - 1957	Va - 1956	Iowa - 1954	Ohio - 1952
	Utah - 1959	R.I. - 1957	Wash. - 1955	Md - 1953	Penn - 1951
W. Va. - 1963	Wyo. - 1959	Ver. - 1958	Wisc. - 1956	Ore. - 1954	Tenn - 1951

Figure A-1. Age in 1966 and date of installation of oldest prestressed bridges in each state in the United States.

Data Not Available:
Ga., Wisc.

0-5	5-20	20-50	50-250	250-500	500-1000	1000-2000
			Ariz. - 92			
			Ark. - 70			
			Colo. - 63			
			Conn. - 140			
			Ida. - 197			
		Dela. - 21	Ill. - 186			
		Hawaii - 30	Ind. - 170			
		La. - 30	Ky. - 50	Iowa - 300		
	Alaska - 14	Md. - 36	Mass. - 200	Mich. - 388		
	D.C. - 12	Nev - 26	Nebr. - 157	Minn. - 300		
	Mo - 10	Ohio - 30	N.M. - 157	Mont. - 450	Calif. - >500	
	N. Hamp. - 16	Okla. - 27	N. Dak. - 94	N.J. - 275	Fla. - ≈ 500	
Ala. - 0	Ver. - 8	P. Rico - 23	N.Y. - 140	S. Car. - 275	Miss. - 700	
Kan. - 2	Wyo. - 7	R.I. - 30	Ore. - 80	Va. - 400	N Car. - 629	Penn. - 1800
Maine - 0	W. Va. - 6	S. Dak. - 43	Utah - 150	Wash. - 350	Tenn. - 700	Texas - 1500

NUMBER OF PRESTRESSED BRIDGES

Figure A-2. Number of prestressed bridges in the United States in 1966.

None	1 or 2	3 - 16	16 - 50	50 - 100	200 - 300
		Alaska - 7			
	D C. - 2	Ariz. - 15			
Ala. - 0	Ill - 2	Conn. - 3			
Dela. - 0	Iowa - 2	Hawaii - 3	Colo. - 20 (est.)		
Ind. - 0	Ky. - 2	Ida. - 10	Fla - 20		
Kan. - 0	Mass. - 2	Miss. - 14	Minn. - 27		
La. - 0	Md. - 1	Mo. - 4	N Dak. - 28		
Maine - 0	Nebr. - 2	Mont - 9	N. Y - 21		
Mich. - 0	Nev. - 2	Ore. - 6	P. Rico - 23		
N. Hamp - 0	N. Car. - 1	R.I - 7	S Car. - 27		
Okla. - 0	N. M - 1	S. Dak. - 9	Tenn. - 35	Ark. - 68	
Ver. - 0	Ohio - 1	Texas - 15	Va. - 20	N.J - 129	
W Va. - 0	Wyo. - 2	Utah - 12	Wash - 35	Penn - 90	Calif. - 250

NUMBER OF POST-TENSIONED BRIDGES

Figure A-3. Number of post-tensioned bridges in the United States in 1966.

CORROSION RESEARCH BY STATE HIGHWAY DEPARTMENTS

California was the only state that was carrying on any appreciable research on the corrosion of steel in concrete. Pennsylvania had a program of periodic inspection in which evidence of corrosion was one of the inspection parameters. Mississippi was experimenting with linseed oil applied to selected girders, a treatment that has been found fairly effective in the sealing of bridge decks (*85*). New Hampshire had applied an epoxy coating to the concrete girders on a bridge near Portsmouth.

SUMMARY

Perhaps the most important conclusion that can be drawn from the survey is that corrosion failure of prestressing steel was not a widespread or serious problem with prestressed bridges in 1966. In fact, only a very few out of the approximately 12,000 bridges in use at that time were reported to show any visual evidence of corrosion of the prestressing steel and, with the exception of the two cases in Michigan, these bridges were all along either the east or west coast. On the other hand, maximum accumulated exposure time in 1966 for even the oldest bridges was only 15 to 16 years, whereas useful service lives of from 50 to 100 years were being planned for by most bridge engineers.

APPENDIX B

LITERATURE REVIEW OF CORROSION OF PRESTRESSING STEEL IN CONCRETE

One of the early tasks of the research program was to conduct a review of the literature to determine how much was already known about the corrosion behavior of prestressing steel in concrete. Because earlier investigations on corrosion of prestressing steel were quite limited, and because the same mechanisms are active with regular reinforced concrete as with the prestressed, the scope of the review was enlarged to include also those phases of the corrosion of reinforcing steel that seemed especially pertinent to the prestressed bridge problem.

In the review that follows no attempt is made to discuss each and every study that has been reported on the corrosion of steel in concrete. Instead, the review is presented in the form of a summary of the present state of knowledge, with pertinent references inserted when needed to indicate information sources.

The summary was prepared specifically for the bridge engineer. The nature of the corrosion process is discussed first. This is followed by a discussion of such factors as the cement matrix as a corrosion inhibitor, effects of chlorides, effect of concrete cover, and the influence of cracks in the concrete. Other facets of the corrosion problem that are reviewed include incidents of corrosion failures in existing bridges, the use of protective coatings, and brittle fractures of the prestressing steel.

ELECTROCHEMICAL NATURE OF CORROSION

It is now recognized that all corrosion processes in metals are electrochemical in origin. For corrosion to occur, a galvanic cell must first be initiated. In such cells electrons are given up by the metal and accepted by the corroding medium, which is usually oxygen.

The separation between the site where metal corrodes, which is the anode of the cell or battery, and the site where the oxygen is reduced (the cathode) may be only a small fraction of an inch, or it can be measured in terms of inches, feet, or yards, depending on the particular environmental conditions that exist.

Corrosion cannot occur without the presence of an electrolyte to carry the corrosion current. Also, it cannot occur without a potential difference or emf developing between the anode and the cathode. Each cell responsible for corrosion has a reversible potential, E, at equilibrium, which can be calculated from the Nernst equation:

$$E = E^0 - \frac{RT}{nF} \ln \frac{a_Q^q a_R^r}{a_L^l a_M^m} \qquad \text{(B-1)}$$

in which E^0 is the standard electrode potential, n is the number of chemical equivalents reacting, R is the gas constant, T is the absolute temperature, F is Faraday's constant, and the a's are the activities of the reactants and products of the cell. The difference between the reversible potentials for each pair of cells provides the driving force for the corrosion reaction.

The rate of corrosion, however, is not governed by the emf of the reaction but rather by the corrosion current as shown by Faraday's law:

$$W = kIt \qquad \text{(B-2)}$$

in which W is the weight of metal corroded, k is the electrochemical equivalent, I is the current, and t is the time.

Three types of corrosion cells can exist when metals are in contact with an electrolyte. These are discussed in the following paragraphs.

Galvanic or Dissimilar Electrode Cells

Dissimilar metals and alloys which are exposed to an electrolyte will produce their own steady-state potentials. Current will flow between these electrodes if they are connected externally and the anode (more active metal) will corrode. Other examples of this type of cell are found between phases of two-phase alloys, between grains of different orientation within an alloy, and between regions of differing residual stresses (e.g., cold work) within a metal or an alloy.

Concentration Cells

Two kinds of concentration cells are known. The more important of the two is the oxygen concentration or the differential aeration cell. If two similar electrodes are placed in an electrolyte where thorough aeration occurs at one electrode location but not at the other, the difference in oxygen concentration will produce a potential difference and cause a corrosion current to flow. This type of cell accounts for pitting under rust and also is responsible for the establishment of active-passive cells leading to pitting of normally passive alloys.

The second kind of concentration cell is the metal-ion concentration cell. This cell results when areas of the same metal or alloy are in contact with solutions which differ in metal-ion concentration.

Differential Temperature Cells

Differential temperature cells are established when a metal with a temperature gradient is exposed to a common electrolyte of uniform concentration. Cells of this type are not too common in reinforced concrete.

Factors Affecting Cell Activity

In all three types of cell, the ratio of the anodic area to the cathodic area has a strong influence on corrosion rate. Increasing this ratio by reducing the cathodic area of the cell diminishes the current flow and hence the rate at which the metal corrodes. Conversely, decreasing the ratio (small anode, large cathode) causes an increased corrosion rate.

Another factor that has a strong influence on the corrosion rate is polarization. Polarization is caused by the formation of a thin nonconducting film on either the cathodic or anodic areas of the metal early in the corrosion process or sometimes by concentration changes of the electrolyte within the cell. Polarization, when it occurs, can virtually stop any further corrosion. Passivation is a form of polarization caused usually by the formation of a nonconducting film. The presence of ions that destroy the passive film greatly accelerates corrosion rates. Also, the conductivity of the electrolyte has an important influence. Other things being equal, the higher the electrical conductivity of the electrolyte, the faster the corrosion rate of the steel.

More detailed discussions of the electrolytic nature of corrosion and of corrosion processes are given by Uhlig (*1*) and by LaQue and Copson (*2*).

CONCRETE AS AN ELECTROLYTE

As already mentioned, corrosion does not occur unless a metal is in contact with an electrolyte. An electrolyte can be any medium that is capable of carrying a current by ionic conduction. Freshly mixed concrete fits this description because it contains ions in solution. The electrical resistivity of concrete when freshly mixed is reported to be as low as 300 ohm-cm (*3*). On curing, the resistivity slowly increases, and when cured concrete is completely dry it becomes, in effect, nonconducting. The ion-forming compounds are still present in the dry mass, however, and when the dry hardened concrete is saturated with moisture, it once again becomes a fairly good electrolyte.

The effect of sodium chloride on the resistivity of concrete was demonstrated by Henry (*3*). Ten hours after mixing, the presence of 2.3% NaCl reduced the resistivity by a factor of 1.8, but the percentage reduction was less with increasing time. At 28 days, for example, the resistivity of concrete made with tap water was 3,150 ohm-cm, whereas that made with 2.3% NaCl was 2,900 ohm-cm.

Higher resistivities were obtained by Tremper, Beaton, and Stratfull (*4*). These authors used ac rather than dc measurements so as to minimize polarization effects during measurement. In an air-dry condition, mortar specimens varied from 270,000 to 660,000 ohm-cm. The resistivities of the same specimens, when moist, dropped to 21,000 to 44,000 ohm-cm. The decrease in resistivity caused by the presence of sea salts was dependent on the moisture content of the mortar. For example, at a moisture content of 7%, the presence of sea salts (0.60% Cl^- expressed as percentage of air-dry mortar) decreased the resistivity from 50,000 to 28,000 ohm-cm. At 9.5% moisture, the resistivity of the salt-bearing and salt-free mortar specimens was the same; namely, 25,000 ohm-cm.

The presence of NaCl or $CaCl_2$ in concrete may increase the corrosion rate of the embedded steel in two ways. First, the current flow in the electrolytic cell is enhanced by the increased electrical conductivity of the concrete, and second, the presence of the chloride tends to destroy the protective passivating-type film that is normally present on the steel. This latter effect is discussed in greater detail in later sections.

THE CEMENT MATRIX AS A CORROSION INHIBITOR

The corrosion rate of steel is negligible in a highly alkaline environment (*5*). Therefore, because the pH of moist concrete is normally higher than 12.0 (*6*), no corrosion would be expected. This absence of corrosion would be achievable, however, only as long as (1) the concrete was well compacted and contained no large cracks, (2) there was a continuous contact between concrete and steel, and (3) the alkalinity of the cement matrix was maintained at a high level.

The alkalinity of wet concrete is due in large measure to the presence of $Ca(OH)_2$ which is formed as a reaction product in the setting of cement (*7*). The alkalinity can be reduced and the protectiveness lost by depletion of the $Ca(OH)_2$ through reaction with carbon dioxide in the atmosphere. This reaction forms the largely insoluble calcium carbonate, thus lowering the pH of the concrete. Fortunately, the reaction proceeds at a very slow rate; also, carbonation probably does not penetrate deeply into dense, crack-free concrete (*7*).

Acid gases such as SO_2 and NO_2 that are present in industrially polluted atmospheres would also be expected to lower the pH of concrete and thereby permit corrosion to be initiated. The rate at which this lowering occurs at various acid gas concentrations has not been studied, but it could be a very important parameter, influencing the useful service lives of bridges erected in industrial areas.

Both NaCl and $CaCl_2$ are known to reduce the alkalinity of concrete. Chloride-free concrete has a pH of 12.5 to 12.8. According to measurements made by Shalon and Raphael (*6*), this value drops when chlorides are present. According to these same authors, corrosion is possible whenever the pH falls below about 11.5.

Tremper, Beaton, and Stratfull (*4*) found the effect of sea salts on pH to be greater for a saturated $Ca(OH)_2$ solution than for a solution prepared from a hydrated portland cement extract. For example, with saturated $Ca(OH)_2$, 3.5 gm of sea salt per 100 ml of solution decreased the pH from 12.8 to 9.7, whereas with cement extract the decrease was from 12.8 to 12.4. This lower change in the case of the cement extract may have been because of formation of calcium chloroaluminate, thus removing Cl^- from solution.

Voids between the steel and concrete were shown by Monfore and Verbek (*8*) to have an important influence on the corrosion of prestressing steel in chloride environments. Voids with dimensions of the order of the diameter of the wire were the sites at which corrosion normally occurred. This observation at least implies that almost continuous contact between steel and the highly alkaline concrete is required to achieve the maximum protection.

EFFECT OF CHLORIDES

Most of the examples of serious corrosion of steel in concrete have occurred when chlorides were present in the environment. As mentioned earlier, the chloride lowers the pH of the concrete. Even more important, however, is that chloride ions tend to destroy the passivity of the steel and, by so doing, make the steel more susceptible to rapid corrosion.

Calcium Chloride Admixtures

Calcium chloride is sometimes added to concrete in small amounts to accelerate hardening. The potential danger of steel corrosion from this practice has been pointed out on numerous occasions. However, at least one investigator, Blenkinsop (*9*), found no adverse effects even with $CaCl_2$ contents as high as 2% by weight of the cement. His tests were conducted over a 5-year period. Dense and porous concrete was used with and without a 2% addition of flake $CaCl_2$. The specimens were cured in forms at 64°F for 24 hr followed by an additional six-day moist-air curing. The reinforcing steel in some specimens was examined for corrosion immediately after curing, whereas the remaining specimens were divided into two groups, one for laboratory exposure (half immersed in water) and one for outside exposure. Examination of the steel was made after 1 month, 3 months, 1 year, 2 years, and 5 years. It was concluded that the use of 2% flake $CaCl_2$ by weight of cement in a dense, well-compacted concrete will have little effect on the degree of corrosion of the reinforcement. However, in a porous concrete, or one which has been badly compacted, the calcium chloride addition will increase the corrosion.

Roberts (*10*) found that corrosion resulting from voids, porous concrete, or inadequate cover is accelerated by $CaCl_2$. He concluded, however, that in a sound portland cement concrete with adequate cover, the corrosion caused by the presence of 2% flake $CaCl_2$ by weight of cement would appear to have no structural significance on the properties of the pretensioning steel.

Veits (*11*) reported tests on a series of concrete beams 2 in. by 4 in. by 7.5 ft long, pretensioned with 0.079-in.-diameter wires. After 2½-year storage in a dry basement, the beams which contained no $CaCl_2$ exceeded the computed load limit when tested in flexure. Companion specimens which contained 2% $CaCl_2$ by weight of cement failed at low load levels; also, the wires in some areas were found to be corroded more than half-way through the diameter.

Evans (*12*) conducted laboratory tests on both prestressed pipes and prisms. Regular portland and sulfate-resisting cement was used. Six to 8 weeks after steam curing the specimens were broken open and the steel was examined. Corrosion was found to occur only when calcium chloride (2% by weight of cement) was present. The type of cement had no apparent effect.

Monfore and Verbek (*8*), after an extensive laboratory study on the effects of $CaCl_2$ additions on the corrosion of prestressing steel, recommended that calcium chloride should not be used as an admixture in prestressed concrete. They also concluded that corrosion of wire in concrete containing $CaCl_2$ is an erratic process, making quantitative evaluation difficult. The corrosion was only slightly, if at all, dependent on the type of cement, and occurred whether the specimens were stored under moist or dry conditions. Most of their tests were of 2-year duration.

Corrosion failures in a prestressed concrete water-supply line for Regina, Sask., Canada, in 1952 can undoubtedly be ascribed to the use of $CaCl_2$ as an additive. The first 84 pipes (36-in. diameter) manufactured for this project contained no $CaCl_2$, and no corrosion of the steel occurred. The mortar used for the remaining 4,400 pipes was the same, except that it contained 1.5 to 3% of $CaCl_2$ by weight of cement. Examination of a representative sample (58 pipes) of the $CaCl_2$-bearing pipes showed corrosion of the steel in every case. Thus, the conclusion that the $CaCl_2$ was the primary cause of the corrosion failures seemed inescapable. The history and analysis of this pipeline failure is described in detail by Monfore and Verbek (*8*).

On the basis of the investigations and experiences cited, it would appear that $CaCl_2$ admixtures might not be dangerous from the corrosion standpoint if sound concrete with adequate cover could be assured. However, because such assurance is not always possible, the recommendation of Monfore and Verbek (*8*) as well as the requirements of ACI 318-63 (*13*) are believed to be entirely valid; i.e., that because of the potential corrosion hazard, calcium chloride should not be used in prestressed concrete.

It is possible, of course, that there is some threshold amount of $CaCl_2$ below which no corrosion would occur because the cement is capable of tying up some of the chloride ions as calcium chloroaluminate. Although several investigators have reported such a threshold, wide variations in reported values exist, and the level (if one exists) has not as yet been established with certainty. In this connection, Monfore and Verbek (*8*) found appreciable amounts of soluble chloride in hardened cement pastes that contained as little as 0.25% $CaCl_2$.

Corrosion of Reinforcing Steel in Marine Environments

Practically all the accounts of corrosion of reinforcing steel in sea water are concerned with the corrosion of steel bars in ordinary reinforced concrete. Although examples can be found of old concrete structures in sea water that are still largely free of any evidence of steel corrosion (*14, 15*), there are many other examples where corrosion damage has occurred. As long ago as 1916 (*16*) a survey was conducted of concrete structures along the United States seaboard. Corrosion was evidenced above the waterline on the majority of the structures inspected. Conclusions reached as the result of the survey were:

1. Corrosion of the steel occurs first, followed by cracking of the concrete. In most cases, corrosion at the base of the crack was not sufficiently severe to cause structural failure.
2. Corrosion occurs only above the waterline, and then only where the concrete is exposed to sea spray.
3. A large concentration of sea salts was always found in concrete with corroded reinforcing bar.
4. The corrosion of reinforcing bar was not related to chemical deterioration of the concrete.

Although later investigations have contributed to a better understanding of mechanisms of corrosions and to types of concrete that offer the greatest protection, the conclusions reached by this 1916 survey are, for the most part, still valid. It is now known that corrosion of steel in concrete marine structures can be minimized, if not completely prevented, through use of concrete that is largely impermeable (*17*). Although permeability tests are not required in current specifications, low permeability tends to be assured by the current requirements on water-cement ratio, richness of mix, and concrete cover.

The importance of concrete permeability was strongly emphasized by the several investigations conducted by the California Division of Highways (*4, 18, 19, 20*) of the very severe corrosion of reinforcing steel that occurred on the San Mateo-Hayward Bridge in the San Francisco Bay area. These studies showed that the concrete in the corroded areas was highly permeable. The excellent work on the electrolytic cell action present in this bridge structure is treated in a later section.

The conclusion reached in the 1916 survey (that corrosion occurs only above the waterline and then only in the presence of sea spray) has been shown to be incorrect by Beaton, Spellman, and Stratfull (*20*). Reinforcing steel from piles removed from the 37-yr-old San Mateo-Hayward Bridge showed appreciable corrosion at the "mud line," which was approximately 10 ft below the average water level. In most cases, however, maximum corrosion of reinforcing steel in pilings has probably occurred above the tide line.

Most laboratory investigations of the effect of sea salts on corrosion of reinforcing steel have been of relatively short duration. Griffin (*21*), after completing an investigation of this type, stated the following:

> In a marine atmosphere, the best concrete, initially free of salt, will not prevent corrosion of its embedded reinforcing steel; it will only buy additional time. Sooner or later, salts from the sea will penetrate the concrete surface sufficiently to convert a weak electrolyte of free moisture to a powerful electrolyte, with resultant corrosion of the steel.

This statement undoubtedly applies equally as well to prestressed bridge members in a marine environment as to ordinary reinforced concrete.

Other Sources of Chlorides

Deicing Salts

The survey of state highway departments (Appendix A) shows that the practice of deicing bridge decks with sodium chloride and calcium chloride is used by almost all states confronted with snow and ice problems. In fact, of the northern tier of states, only Montana, New Jersey, and Oregon used no deicing salts. The use of NaCl and $CaCl_2$ for deicing roadways increased noticeably in the middle 1950's, mostly as a result of a demand from motorists for safe, all-weather driving conditions. The experience with deicing in Wisconsin is probably typical of most northern states. Prior to 1956 only abrasives were used, together with a small amount of salt to prevent freezing in stockpile (*22*). In 1956 a "bare pavement" policy was adopted. This called for the liberal use of deicing salts. Straight rock salt (NaCl) is now applied when the temperature is 25°F or higher, and NaCl-$CaCl_2$ mixtures are applied when the temperatures are below 25°F. The chemicals are applied at an approximate rate of 400 lb per mile, depending on conditions.

A relationship appears to exist between the increased use of salts and the increasing incidence of scaling and spalling of concrete bridge decks (*22, 23, 24, 25*). The same trend has been observed in the corrosion of automobile bodies (*26*). Chlorides have become the most widely accepted method of deicing (*27* and Appendix A) and will probably continue to be used on bridge decks until some better and more economical method is discovered. Thus, this source of chlorides in prestressed concrete bridges must be considered as important with respect to steel corrosion. Water containing deicing salts could seep through cracks and seams in the deck and come into contact with the prestressed bridge girders, or traffic splash containing chlorides might, under some circumstances, reach the prestressed structural members.

Chlorides from Soils

Chlorides are present in some soils in arid and semi-arid regions. However, because of the lack of moisture in such regions, corrosion of embedded steel would not appear to be a problem. Also, chlorides from soils would not present a very serious problem to bridge superstructures; the corrosion dangers from this source of chloride would come mostly in abutments and pilings that were in intimate contact with the soil. Dragging of prestressing steel over such soils during construction, however, could be a source of later corrosion difficulties.

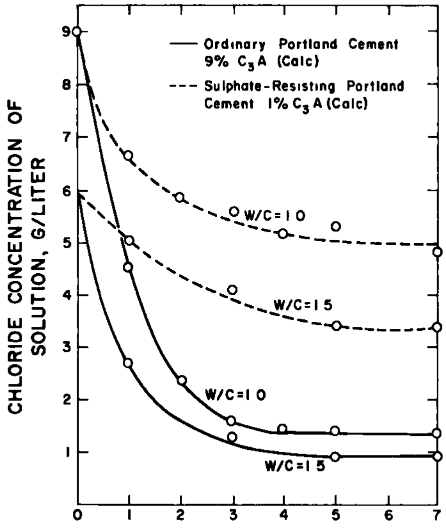

Figure B-1. Chloride solubility at 25°C of portland cement mixtures prepared with 1.4% by weight $CaCl_2$ (10).

Chlorides from Cement, Aggregates, and Mixing Water

Chlorides can be present in small amounts in cements, aggregates, and mixing waters. Because chloride from these sources could initiate corrosion of the steel, Dumas (*28*) has proposed that the combined chloride content be limited to 0.05%, based on the weight of the cement. Schultz (*28*) indicates an even lower value of 0.03% for the aggregates and the mixing water. In Western Germany the permissible amounts of chloride are 0.10% for the cement and 0.03% for the mixing water, whereas the aggregates are allowed to contain 0.02% of soluble chloride (*28*). California permits not more than 650 ppm Cl^- in water for prestressed concrete (*29*). This reduces to 0.026% Cl^- by weight of cement in a mix with a W/C ratio of 0.4.

The view that small amounts of chlorides can be tolerated in concrete presupposes that minor quantities of chloride will be fixed by the hydrated cement and therefore rendered harmless. Insofar as could be determined, the actual limits that have been set on maximum permissible amounts of Cl^- appear to be based more on personal opinions than on corrosion experiments.

Reaction of Chlorides with Concrete

It has been known for many years that portland cements react with chloride solutions and, by so doing, remove the chloride ion from solution (*30*). The reaction, which is between hydrated C_3A and the chloride ion, results in the formation of the relatively insoluble calcium chloraluminate, $3CaO \cdot Al_2O_3 \cdot CaCl_2 \cdot 2H_2O$ (*31*). Hence, the higher the C_3A content of the cement, the greater its capacity for removing chloride from solution. Roberts (*10*) demonstrated this effect for two cements of different C_3A content. Figure B-1, which is taken from Roberts' data, shows that ordinary portland cement with a C_3A content of 9% is considerably more effective in tying up chloride than is a cement with a C_3A content of only 1%. Somewhat similar results were obtained by Monfore and Verbek (*8*).

This reaction of the cement with chloride ions undoubtedly has a strong influence in determining the rate at which chloride will penetrate concrete, especially in those cases where the supply of chloride at the surface is limited. It is obvious that chloride ions that react near the surface to form calcium chloroaluminate will not be free to penetrate further into the concrete.

CHLORIDE PENETRATION INTO CONCRETE

Laboratory Studies

Ost and Monfore (*32*) investigated the penetration of $CaCl_2$ into various concretes and cement pastes. Aqueous solutions containing 2% and 8% $CaCl_2 \cdot 2H_2O$ were kept in contact with the top surface of 2-in.-diameter by 4-in.-high concrete and cement-paste cylinders. The curved surfaces of the cylinders were sealed with plastic sheaths. Penetrations were related to time of contact, richness of mix, and water content. Penetration was more rapid into concrete than into cement paste. Type 1A cement was used for all tests.

Figure B-2 is a plot of the measured $CaCl_2 \cdot 2H_2O$ content at three depths after various times of exposure. The data points were taken from Figure B-3 of the Ost and Monfore paper and replotted to better illustrate the effects of exposure time. Note that the chloride content of the concrete at all depths is increasing with time. The amount of $CaCl_2 \cdot 2H_2O$ in the concrete varied greatly with water-cement ratio and the cement content of the concrete as shown by the following data reported for a depth of 2 in. after an exposure time in 2% $CaCl_2 \cdot 2H_2O$ of 1 yr (Table B-1).

The amount of calcium chloride that had penetrated 2 in. into concrete having a water-cement ratio of 4.4 gal per bag and cement content of 7.0 bags per cubic yard after exposure to 8% $CaCl_2 \cdot 2H_2O$ for 1 yr was only 0.1%. However, according to the authors, after exposure for many years this value would probably be significantly larger.

Ost and Monfore (*32*) also studied the reduction of tensile strength from corrosion of prestressing wires 0.162 in. in diameter, embedded in concrete prisms prepared with various water-cement ratios. The specimens were designed to provide 0.5 in., 1 in., and 1.5 in. of concrete cover. The wires were not stressed. Exposures were made either in water or in a saturated $CaCl_2$ solution by means of towels that were draped over the prisms and extended down into the liquid. No corrosion was found in any of the specimens exposed for 2 years in water. Corrosion and reduction in tensile strength were observed in the $CaCl_2$ environment,

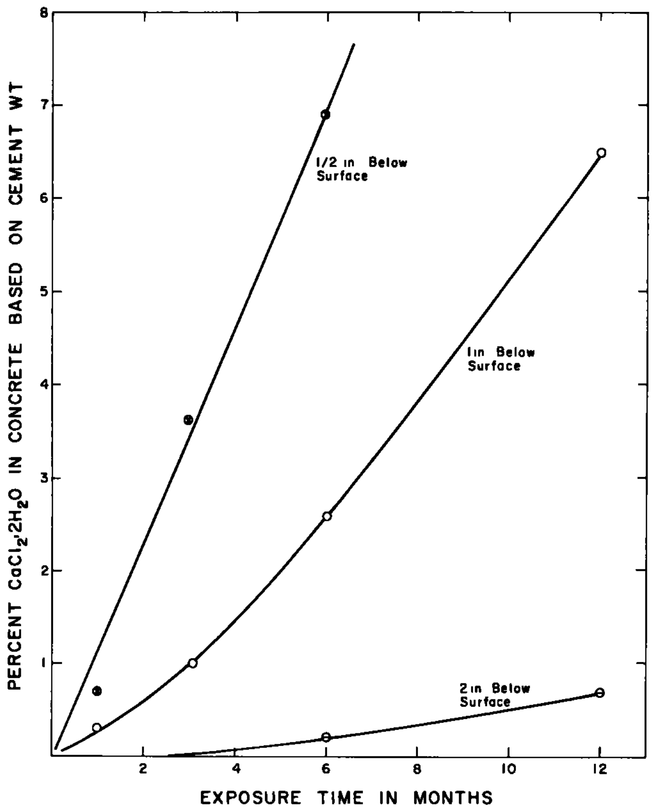

Figure B-2. Preparation of 2% by weight $CaCl_2 \bullet 2H_2O$ solution into concrete specimens prepared with 5.5 bags cement per cu yd and with 5.2 gal of water per bag of cement (32).

however, for concretes having water-cement ratios of 7.8 and 5.7 gal/bag but not for concrete having a ratio of 4.7 gal/bag. Hence, these results are in agreement with their $CaCl_2$ penetration study in that chloride did not penetrate (as indicated by absence of steel corrosion) into the dense, low water-cement ratio concretes. Again, exposure time was limited and conclusions cannot be drawn as to the extent of penetration that might have occurred after extended years of exposure.

Chloride Penetration into Bridge Decks

Many valuable data on penetration of chlorides are included in two recent reports published on the durability of concrete bridge decks (*24, 25*). The chloride analyses in these two studies were made on cores removed from "in-service" decks. The reason for the analyses was to determine if a relation existed between the chloride content of the concrete and the observed scaling and spalling of the deck surface.

One report (*24*) gives the results of chloride analysis at different depths on 124 cores taken from 18 bridges in Kansas. Chloride was detected in each of the 124 cores, the amounts normally decreasing with distance from the bridge deck surface. The maximum amounts detected varied from 0.50% chlorides by weight of mortar near the surface to 0.19% 2.75 in. below the surface. For comparison with the work of Ost and Monfore (*32*), these two values convert to 1.9% and 0.4% by weight of $CaCl_2 \cdot 2H_2O$, respectively.

Similar results were obtained from analyses of 60 cores from 13 bridges in Michigan (*25*). The maximum amounts of chlorides detected in this study varied from 0.62% (by weight of mortar) near the surface to 0.15% 2.75 in. below the surface.

The importance of these studies from the standpoint of steel corrosion comes in showing that deicing chlorides added to the surface of a bridge deck will, in time, penetrate the concrete to appreciable depths. Similar penetration behavior might be expected from solutions of deicing salts that came into contact with the surfaces of prestressed girders from some such cause as splashing of road slush or leakage into the superstructure through cracks and seams in the deck.

Chloride Penetration into Pilings

Tyler (*33*) investigated five principal cement types for concrete pilings exposed in three sea-water and one fresh-water location. The piles, which were 12 in. square, were reinforced with ⅝-in. round steel bars positioned so as to provide a 1.5-in. concrete cover. Inspections after 15 to 17 years showed that the 1.5-in. cover was insufficient to prevent rusting of the steel and the subsequent cracking of the concrete. Concretes with the highest cement contents and lowest water-cement ratios showed the best performance, but some corrosion was observed for all concretes.

Lea and Watkins (*34*), after laboratory experiments with partially immersed simulated concrete pilings, concluded that the primary cause of concrete deterioration was corrosion of the reinforcing steel. Beaton, Spellman, and Stratfull (*20*) examined 17 reinforced concrete piles from a

TABLE B-1

VARIATION OF $CaCl_2 \bullet H_2O$ IN CONCRETE

CEMENT CONTENT (BAGS/CU YD)	WATER CONTENT (GAL/BAG)	AIR CONTENT (%)	% $CaCl_2 \bullet 2H_2O$ BY WEIGHT OF CEMENT AT 2-IN. DEPTH
4.0	7.0	6.8	4.4
5.5	5.2	5.0	0.6
7.0	4.4	4.2	0.0

37-year-old bridge over San Francisco Bay and found that 47% of the piles showed significant corrosion of the reinforcing steel, with average pit depths of 0.114 in. Analysis of the chloride content of the concrete near the pitted steel showed as high as 33.6 lb of chloride per cubic yard of concrete. No relation was observed between chloride content and degree of corrosion of the steel. Chlorides, however, were high in all samples (19.2 to 33.6 lb per cubic yard of concrete).

Figure B-3 shows the degree of penetration of chlorides into one of the pilings of this same bridge after 28 years, as reported by Gewertz and Tremper *et al.* (*4, 57*). The core used for the analysis was taken from above the waterline. The chloride contents plotted are from 1-in.-thick disks cut from a 2-in.-diameter core. It is obvious that after 28 years some chloride has penetrated as far as 5.5 in. into the concrete.

Summary of Chloride Penetration

From a review of the literature it would appear that four factors affect the rate of penetration of chlorides into concrete: (1) the density and/or degree of imperviousness of the concrete, (2) the availability of chlorides at the concrete surface; i.e., whether there is continuous or intermittent contact, (3) the time of contact, and (4) the tricalcium aluminate content of the cement used for preparing the concrete.

EFFECT OF SALTS OTHER THAN CHLORIDES

According to present knowledge, only halogen ions (of which only chlorides are of practical significance to concrete) are capable of penetrating and destroying the passivating layer on steel embedded in concrete (*28*). The effect of sulfides and nitrates, which are potentially dangerous from the standpoint of brittle fracture of prestressing steel, are discussed in a later section.

CORROSION CELLS IN CONCRETE

That electrochemical cells are, in fact, active when steel in concrete is corroding has been shown by studies on buried reinforced and prestressed concrete pipelines (*35*), on prestressed concrete tanks (*36*), and also on one reinforced concrete bridge over sea water (*4*). For the bridge, a plot of potential profiles showed that corrosion of the steel was occurring only at the anodic areas. These anodic areas were found, in some cases, to be separated from the cathodic regions by distances of from 2 to 10 ft.

Stratfull (*18, 19*) explained the corrosion of the bridge as being due to unequal salt concentrations in the concrete which, in turn, generated differential salt concentration cells. Lewis and Copenhagen (*37*) also ascribed corrosion of reinforcing steel in concrete structures in coastal regions of South Africa to differences in salt concentration. A cell of the following type was pictured:

Steel (Anode)	Permeable concrete	Less permeable concrete	Steel (Cathode)
	Low pH High Cl$^-$	High pH Lower Cl$^-$	

These authors stated, however, that no corrosion would occur unless oxygen was present.

Several other investigators have contended that the corrosion of steel in concrete is caused by differences in oxygen availability at the steel surface. Finley (*17*) has cited extensive evidence to support his view that oxygen concentration cells cause most of the corrosion of steel in concrete. The investigations of Scott (*38*) and Hausmann (*35*) demonstrated that, in simulated chloride-containing concrete environments, corrosion of the steel does not occur unless bubbles containing oxygen are trapped against the steel surface. This trapping could conceivably cause oxygen concentration gradients along the steel. A "thin-film" hypothesis was proposed by Scott (*38*) to account for the observed differences in corrosion behavior between trapped bubbles and dissolved oxygen. According to this hypothesis, a monomolecular layer of water on the steel surface beneath the bubble permits continuous replenishment of oxygen lost in the corrosion process.

Evidence that oxygen-gradient cells are mostly responsible for corrosion of steel in concrete is given in Appendix D.

STRESS CORROSION AND HYDROGEN EMBRITTLEMENT

Brittle fracture of prestressing steel by either stress corrosion or hydrogen embrittlement is especially dangerous from the safety standpoint because such fractures can occur with

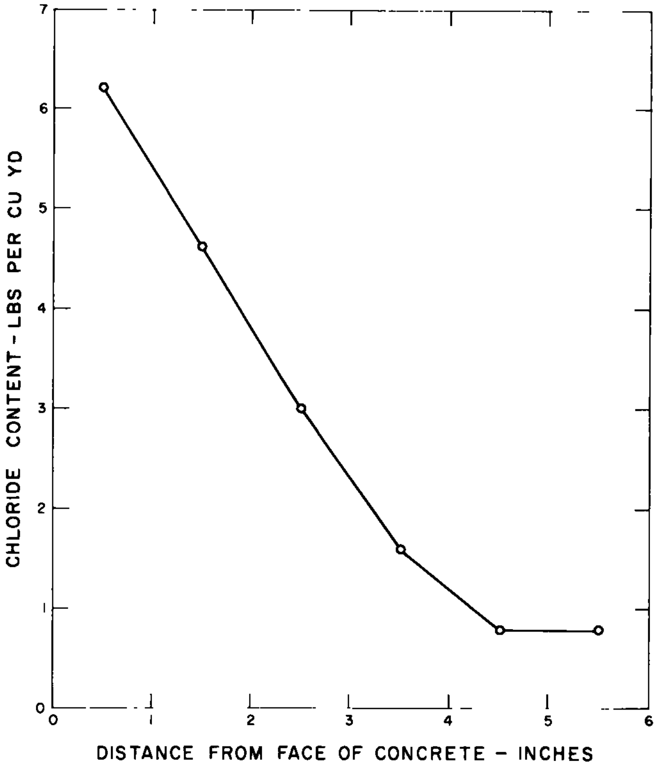

Figure B-3. Chloride content of a concrete core taken above the waterline from a bridge pile exposed 28 yr in San Francisco Bay (4).

no prior observable macroscopic deformation of the steel and with no advance warning.

There is a need for clarification of concepts of hydrogen embrittlement and stress-corrosion cracking with respect to how and where these phenomena occur in prestressing steel. It is true that hydrogen which is cathodically charged into stressed steel in an electrochemical cell can result in brittle failure of the steel; however, these failures are not necessarily stress-corrosion failures, and, conversely, stress-corrosion failures are not classified as being due to hydrogen embrittlement. Another area which should be clarified is that of hydrogen blistering, as compared to hydrogen embrittlement. The sections that follow present an attempt to define these important mechanisms and phenomena.

Hydrogen Embrittlement

The embrittlement of steel by hydrogen is characterized by the sensitivity of the embrittling phenomenon to slow strain rates and to elevated temperatures. This type of sensitivity suggests that the embrittlement is controlled by the diffusion of hydrogen. According to Troiano (*39*), a steel which has been embrittled by hydrogen shows the following characteristics:

1. The notch tensile strength may be less than normal and directly reflects the loss of ductility due to H_2.
2. Delayed failure may occur over a wide range of applied stress.
3. There is only a slight dependence of the time-to-failure on the applied stress.
4. There is a minimum critical value of stress below which the failure does not occur.

The question of possible hydrogen embrittlement of galvanized steel in prestressed concrete was raised by Bird, at least indirectly, when he reported that zinc will evolve hydrogen when embedded in concrete (*40*). His studies showed that a drop in potential of zinc occurred at pH values of from 12.6 to 12.7, which coincided with the beginning of hydrogen evolution.

The catastrophic failures of steels which are stressed in tension in aqueous environments containing H_2S have been classified as hydrogen embrittlement failures. It is a well-known fact that the corrosive action of H_2S on steels produces hydrogen and that the sulfide ion acts to poison the steel surface to allow penetration of atomic hydrogen into the steel lattice. Generally, it can be stated that steels with a hardness greater than R_c25 are susceptible to H_2S stress cracking. However, recent research investigations indicate that for certain steels there appears to be no hardness below which failure will not occur under very severe conditions of stress, cold work, and high H_2S concentration (*41*).

Hydrogen Blistering

Atomic hydrogen which enters a steel by any means—whether by cathodic charging during electroplating or while under cathodic protection or as a result of a corrosion process—may recombine within the steel to form molecular hydrogen whenever a void or an interface is encountered. For example, rimmed steels have been found to be extremely susceptible to the formation of hydrogen blisters when used as vessels in the petroleum industry and exposed to H_2S-containing crude oils (*42*).

Stress-Corrosion Cracking

As the term implies, stress-corrosion cracking (SCC) is the result of a combined stress and corrosion state. However, in consideration of the stresses involved, it is found that the residual stresses are usually of more consequence than the applied stresses. Also, the amount of corrosive attack which occurs prior to cracking failure is often not measurable. It is significant, too, that a specific ion species in the corrosive environment is required before failure by SCC occurs.

Many mechanisms of stress corrosion have been proposed, but at present there is no adequate theory that will explain the mechanism completely. Generally, a threshold stress exists below which SCC does not occur, and the presence of this threshold stress has been demonstrated for most alloys which have been found susceptible to SCC.

Logan (*43*) lists the environments which result in SCC of various iron-base alloys. He lists the following for ordinary mild steel:

1. $NaOH + Na_2SiO_3$;
2. $Ca(NO_3)_2$, NH_4NO_3 and $NaNO_3$;
3. $HCN + SnCl_2 + AsCl_2 + CHCl_3$;
4. Na_3PO_4;
5. Pure NaOH;
6. $NH_3 + CO_2 + H_2S + HCN$;
7. NaOH, KOH solutions;
8. Monoethanolamine solution $+ H_2S + CO_2$;
9. $Fe(AlO_2)_3 + Al_2O_3 + CaOH$ solution;
10. $HNO_3 + H_2SO_4$;
11. $MgCl_2 + NaF$ solution;
12. Anhydrous liquid NH_3;
13. H_2S media; and
14. $FeCl_3$ solution.

Of this list, only No. 13 (H_2S) might be expected to be present on certain rare occasions in the environment of a prestressed bridge.

Tests were performed by Kajfasz and Czerniak (*44*) on 0.197-in.-diameter wires, cold drawn to 242 ksi, to determine the effect of torsional stresses in wire on corrosion resistance, especially stress corrosion. A solution of 5N ammonium nitrate was used as the corrodent, with approximately one-half of the specimen length immersed. Initial stress levels of 136 ksi to 171 ksi were applied, and a condition of constant elongation was maintained. Relaxation of the specimens was also accounted for. Twists of 180°, 360°, and 3 × 180° per linear meter were applied to some specimens prior to the superposition of the axial tensile stress. Some specimens were given a permanent torsional strain. At the termination of the tests, it was concluded that torsional stresses introduced in cables for prestressed concrete by the stranding process may constitute a factor which increases the sensitivity of the wire to stress corrosion. The twist of a wire reduces the time to failure in the corrosive environment. Assuming the most serious test results as critical values, the lifetime for wires was 0.70 for a 360° twist and 0.33 for the 3 × 180° per linear meter twist, as compared to unity for non-twisted wire. These data serve to emphasize the importance of stress relieving after the stranding operation.

Bukowiecki is mentioned in a recent review (*45*) as having performed tests on tensioned wires that were either periodically wetted or continuously submerged. He concluded from this work that stress corrosion of cold-drawn steel in concrete contaminated with chlorides was not a problem. The same review mentions also stress-corrosion tests in a boiling solution of calcium and ammonium nitrate as conducted by L. H. Everett. Another aspect of stress corrosion discussed was hydrogen sulfide stress cracking resulting from high alumina content cements.

Tremper (*46*) quotes a section of an ACI report, "Admixtures for Concrete," by Committee 212, on the possibility of stress-corrosion cracking of prestressing steel because of the presence of calcium chloride. However, no justification for the statement is given.

Unz (*47*) developed an electrolytic method for stress-corrosion tests of prestressing steel which he claimed predicted the susceptibility to SCC of high-tensile steel in prestressed concrete. This test, however, is not very satisfying, and appears to be based on very few experimental runs.

Gilchrist (*48*) investigated samples of wires from several prestressed concrete pipes and storage tanks in which failures had been attributed to stress-corrosion cracking. Boiling nitrate tests of stressed wires taken from two sites (Regina pipeline and Owls Head sewage tank) did not produce cracking failures. It was found, however, that occasional longitudinal cracks were present in the wires taken from the concrete. Gilchrist assumed that these cracks formed during the exposure period. Examination of microstructures showed that where no longitudinal cracks existed the structure was entirely pearlitic, but that in areas adjacent to cracks particulate carbides were present in the pearlitic matrix. It was concluded that prestressing steel that is entirely pearlitic is not susceptible to stress corrosion. Gilchrist, however, apparently overlooked the possibility that the longitudinal cracks were present in the wires before use. Occasional cracks of this type were observed in studies described in Appendix C for as-received wires that had never been subjected to corrosive testing.

EFFECT OF CONCRETE COVER

It is obvious that, with the same concrete, the greater the thickness of concrete cover the longer it will take for corrosive solutions to reach the steel. Thus, the thicker covers should be more effective in inhibiting corrosion. This appears to be valid below about 2 in. of cover, but beyond 2 in. it has been found that the degree of corrosion protection cannot be increased significantly (*49, 50, 51*).

The reasoning of the greater the cover the greater the protection is present in many concrete specifications. For example, the American Association of State Highway Officials (*52*) requires 1.5-in. cover over prestressing steel, except that "in locations where members are exposed to salt water, salt spray, or chemical vapor, additional cover shall be provided."

A view held by some investigators is that the relatively heavy covers required by AASHO are not necessary for corrosion protection. According to Abeles and Filipek (*53*) a cover of only 0.5 in. is considered sufficient in several European countries, even though the construction is exposed to chemical attack. They do, however, stress the need for high-density concrete if the small covers are used. They also state that the designer must bear in mind the practical difficulties of placing and compacting high-density concrete if small covers are used and that, in practice, the cover selected will be a compromise between working conditions and the minimum permissible cover as indicated by research tests. In a later paper (*54*) Abeles states that too small a cover (for example, 0.25 in.) is insufficient if the concrete is exposed to outside weathering because, under these conditions, the protectiveness of the thin concrete layer could be destroyed by carbonation.

Griffin and Henry (*55*) exposed experimental walls with reinforcing grids of 0.25-in. prestressing steel and of ⅝-in. reinforcing bar to a daily 5-min application of sea-water spray for a period of 2½ years. With a 1-in. concrete cover, rust staining appeared in about 15 months, followed by cracking some 5 months later. With a 6-in. cover, the first signs of incipient corrosion were not detected until about 2½ years after the testing started (*21*). Thus, the greater thickness of cover certainly delayed salt penetration and steel corrosion but, even with a cover of 6 in., corrosion was beginning to occur after 2½-year exposure to the test conditions.

The permeability of the concrete cover is obviously important because it governs the rate at which aggressive fluids can penetrate to the steel. A concrete cover of 0.5 in. with extremely low permeability could easily provide more protection than a 2-in. cover of a porous permeable concrete. Permeability limits in specifications based on actual measurements would, of course, prevent the use of permeable types. However, it may not be necessary to go to such lengths, inasmuch as low permeability can almost always be assured by control of cement type, richness of mix, water-cement ratio, placement and compaction, and compressive strength. Requirements on these variables are presently incorporated into most specifications. Most data are needed, however, on the effects of these variables on the permeability of the hardened concrete to corrosive solutions, and more especially on the permeability of oxygen and carbon dioxide. Griffin and Henry (*55*) have obtained data on water vapor transmission. The rate of transmission was found to be largely independent of specimen thickness. However, different transport mechanisms are probably active for the case of O_2 and CO_2.

EFFECT OF CRACKS IN CONCRETE ON STEEL CORROSION

Cracks that extend from the surface of the concrete down to the reinforcing steel constitute a potential corrosion hazard because they permit easy access for air, moisture, and corrosive solutions such as chlorides.

Cracks are quite common in ordinary reinforced concrete structures. In prestressed girders, however, cracks very seldom appear, because in most prestressed members the concrete is either under compression or under only a moderate tension. Nevertheless, it is possible to obtain cracking in prestressed concrete from such factors as (1) shrinkage of the concrete, (2) accidental overload, (3) unsuitable aggregates, (4) steel corrosion, and (5) frost action.

Shrinkage of the concrete is probably the most important cause of cracks in prestressed concrete. Cracks of this type are normally controlled by mild steel reinforcing and, even if present, they seldom penetrate to the prestressing steel. In addition, they tend to be present as very narrow hairline cracks.

In the case of ordinary reinforced concrete, the ACI Standard Building Code (*54*) requires that "... the average crack width at service load at the concrete surface of the extreme tension edge, does not exceed 0.015 in. for interior members and 0.010 in. for exterior members." Neither the maximum crack width nor the crack width at the level of the reinforcement is mentioned, yet these two parameters would seem to be highly important. The European Concrete Committee has suggested that the maximum permissible crack width at the reinforcement should be 0.012 in. for interior structures, 0.008 in. for exterior locations, and 0.004 in. for structures exposed in aggressive environments (*54*).

With prestressed concrete, cracks become more important since, because of the smaller size of the steel and the higher stress levels, corrosion presents a more serious threat. This possible corrosion danger appears to have been the main reason for designing prestressed girders so that the maximum tension in the concrete does not exceed 250 psi (*52*). With high-strength concrete, no cracks should appear at this stress level. A more economical design would be one in which some cracking of the concrete was permitted under live load conditions. Abeles (*53, 54*) has been a strong proponent of this "partial prestress" concept. Tests made under his supervision showed that micro-cracks which were invisible to the unaided eye did not permit corrosion of tendons after 3 months' exposure to sulfur dioxide fumes (*54*). Other tests were made in which two small beams were exposed for 1 year in the same environment. The beams were live loaded as a pair so as to produce one crack 0.004 in. wide in each beam. With 1-in. cover, no corrosion of the steel could be detected after the 1-year exposure. With a 0.5-in. cover, however, some corrosion occurred, as evidenced by a slight staining of the steel in the vicinity of the crack. With no concrete cover, steel corrosion was severe, the maximum reduction of the wire radius due to pitting being 11.2%. In tests with steam and sea water on prestressed members with artificially opened cracks, Abeles found that crack widths of 0.01 in. appeared to be harmless from the standpoint of steel corrosion if dense concrete was used (*53*).

Hausman (*56*) stated that minor cracks penetrating to the steel are normally not damaging and that they frequently heal in a moist environment. Gewertz (*57*) concluded that the steel corrosion in the San Mateo-Hayward Bridge in California was independent of surface cracking. Tremper (*46*), after a review of the literature, stated the following: "As a practical matter, therefore, it does not appear that cracks in concrete are of decisive effect with respect to the development of corrosion."

In contrast to these various observations on the absence of serious corrosion at cracks, extensive laboratory and field investigations under different climatic conditions that were conducted by Rehm and Rauen (*58*) showed that corrosion of the reinforcing steel could occur at a crack width as small as 0.004 in. These authors summarized their findings as follows:

> The corrosion is most pronounced for beams being exposed to the polluted air of industrialized areas. For crack widths up to 0.10 mm (0.004 in.) either no corrosion or in rare instances only slight corrosion was observed. For crack widths between 0.10 mm and 0.25 mm (0.004 and 0.010 in.) the frequency of corrosion increased rapidly and in some cases heavy corrosion took place. For a crack width greater than 0.25 mm (0.010 in.) the reinforcement was always corroded.

These findings are in agreement with results of at least two other investigations (*57, 58*).

In summary, it seems safe to conclude that cracks in concrete that extend to the steel and that are sufficiently wide will permit corrosion of the steel to occur. Contradictions exist as to maximum permissible crack width. This is understandable because the various tests were not made in the same way or under identical environments or for the same time durations. According to Rehm (*28*), the extent of damage due to corrosion at cracks is always time-dependent. Thus, even microcracks, if given sufficient time, might permit corrosion to occur. In Rehm's opinion, the ideal prestressed concrete structure from the corrosion standpoint would be one that remained free of cracks throughout its entire service life (*28*).

INCIDENTS OF CORROSION FAILURES OF STEEL IN PRESTRESSED BRIDGES

Information as to corrosion failures of prestressing steel in bridges is quite limited. The survey of state highway departments (Appendix A) showed that evidence of steel corrosion is extremely rare and that actual failure of tendons had occurred in only one of the thousands of highway bridges that are now in service. Nevertheless, some examples exist where corrosion failures have occurred outside of the United States. These failures are explainable, however, and could have been avoided by proper use of materials, proper detailing, and proper construction methods.

Details for most of the failures cited subsequently are lacking. Also, most of the information was obtained on a confidential basis; therefore, the source of the information and the exact geographical location are sometimes omitted.

Van Loenen and Etienne (*61*) reported an unusual case of tendon failures in a bridge constructed "somewhere in the Netherlands" in 1964-65. The bridge was post-tensioned with straight (undraped) cold-drawn and stress-relieved wires. The wire, which was 0.275-in. in diameter, had a minimum UTS of 242,000 psi. Prestressing was done at 162,000 psi.

The sheath of the button-headed wires (B.B.R.V. system) consisted of an aluminum trumpet attached to a mild steel duct.

Portland cement grout with no additives was used during summer construction and no difficulties were encountered.

During the early winter, however, 1% by weight of sodium carbonate was added to accelerate hardening. Occasional wire failures then occurred within a few days.

Subsequent research by these authors suggested that the failures were the result of hydrogen embrittlement. Their explanation was that the contact between aluminum and mild steel formed a cell in which the aluminum was the anode and the steel the cathode. When sodium carbonate was present in the grout, the cell became active, liberating hydrogen at the steel cathode. With only small amounts of sulfides present in the grout (0.05%) to serve as a "poison," the hydrogen entered the steel and embrittlement failures occurred within a few days.

Vandepitte (*62*) reported some long-range deterioration of a post-tensioned bridge in Denmark. The bridge, which was located above a railroad yard, had external tendons. The pitting-type corrosion of the steel was attributed to high humidity, combined with the aggressive gases from steam locomotives.

The German Federal Railroads (*63*) is currently investigating the corrosion damage to prestressing steel in overpasses above railroad tracks. The serious corrosion sometimes observed was attributed to the chemical interaction of sulfides in the high alumina cements with gases from the steam locomotives.

Several examples of corrosion difficulties in post-tensioned bridges are included in an appendix of a recent in-house report of the Jersey Central Power and Light Co. (*64*). One was a floating bridge over sea water in which 1⅛-in.-diameter ungrouted high-strength bars were used as temporary tendons. Failures, which occurred within 2 months, were ascribed to stress corrosion by exposure to the severe marine environment.

Another example cited was a cantilever repair on a bridge pier. The repair was made using ¼-in.-diameter wire as an external tendon. Gunite to a thickness of approximately 1.5 in. was applied for protection. After 5 years, rust staining was detected on the shotcrete and the tendons were replaced. The corrosion was ascribed to water with deicing salts running off the bridge deck into a separation caused by poor bond of shotcrete to the pier face. This lack of bond permitted the runoff water to seep into the separation and reach the tendons.

Yet another example was a post-tensioned bridge over sea water which connected coral reefs. Post-tensioning was done with 1.25-in.-diameter high-strength bars placed in flexible galvanized ducts. The concrete was prepared using 2-in. coral aggregate. Some corrosion of the bars was noted after 8 years. Corrosion of stirrups, which had only 0.75-in. cover, occurred first. This caused spalling of the concrete, which exposed the galvanized duct. The duct corroded through, thus exposing the bottom bars of the girders to corrosion. The grout was apparently not sufficiently protective once corrosion had perforated the duct wall, because the bottom bars showed pit depths up to 1/16 in.

Corrosion difficulties with another post-tensioned bridge over sea water are also described in the Jersey Central Power and Light report. In this bridge, the ducts were placed flush with the bottom of the girder. Spalling of concrete occurred at abutment spans after 14 years.

Xercavins (*65*) reported that in France a pretensioned bridge erected in a marine atmosphere must be rebuilt in the near future after having been in service for only 15 years. The web thickness of the girders was only 5.5 in. Concrete cover was "insufficient" and there is evidence of poor compaction of the concrete. Rust staining of the concrete occurred and this was followed by a "severe" cracking along the cables. Xercavins states that all of the evidence points to the type of corrosion that can be expected when the regulations regarding minimum cover are not respected.

It is believed highly significant that, up to the present, not one catastrophic failure has been reported for a prestressed concrete bridge. Furthermore, for those rare cases in which serious steel corrosion has occurred, the corrosion could have been avoided by the application of proper design and construction practice.

USE OF COATINGS FOR CORROSION PROTECTION

The potential danger of long-time corrosion causing failure of prestressed members has led to some consideration of coatings to protect the prestressing wires. The coating requirements for prestressing wires are much more exacting than for ordinary reinforcing bar because, for prestressing wires, the coating must be able to withstand the elongations that occur during tensioning, which may be as high as 0.7%.

Few brittle coatings can withstand elongations of this magnitude without cracking. Coating cracks, if they occur and if they are of sufficient width, would permit corrosion to occur and thus defeat the purpose of the coating. The only exception would be a coating such as zinc, where cathodic protection might be expected to inhibit corrosion in the event that cracking occurred during prestressing.

In the case of pretensioned concrete members, any coating that was applied would first need to provide a good bond to the concrete and, second, the coating would need to be sufficiently creep-resistant to maintain the high stress level in the wire for the long periods of use expected of pretensioned members.

Most of the work on coatings reported in the literature has been directed to the protection of ordinary reinforcing bars. This work is considered in the present review, but the main emphasis is placed on coatings for prestressing steel.

Metallic Coatings

Zinc Coatings

The possibility of protecting steel reinforcing bars from corrosion by means of a zinc coating is not a new concept. The interest dates back to at least 1918, when several types of zinc coatings were investigated for possible use in concrete barges (*66*). Halstead (*67*) examined a galvanized bar encased for 54 years in concrete exposed in marine air and found that rusting of the bar had occurred at only a few isolated spots. Benjamin (*68*) reported that galvanized sheet steel corroded less than uncoated sheet in a concrete containing 1% $CaCl_2$. Mange (*69*), on the other hand, reported severe corrosion of galvanized sheet when excess water was used to prepare concretes containing $CaCl_2$ admixtures.

According to Woods (*70*), the primary reaction between zinc and calcium hydroxide is

$$Zn + Ca(OH)_2 \rightarrow CaZnO_2 + H_2$$

Woods states that the probable chemical reaction when $CaCl_2$ is present is

$$ZN + CaCl_2 + 2H_2O \rightarrow 2CaZnO_2 + 2HCl + H_2$$

In either reaction the products of reaction are not voluminous and, therefore, damaging stresses are not created within the concrete. The usefulness of a zinc coating has not been demonstrated for a marine environment.

Cornet and Bresler (*71*) compared the properties of ungalvanized and galvanized prestressing wires that had an initial diameter of 0.148 in. Both coated and uncoated wires were from the same lot of steel. Table B-2 summarizes the test results.

Tests with concrete beams were also conducted. The beams were 3 in. by 5 in. by 6.25 ft long. Galvanized wires were employed for prestressing two beams and uncoated wires for a second two. All four beams were stressed at 125 ksi, based on the area of the uncoated wire. The force on the wires was transformed from the forms to the concrete after curing for 7 days. After 28 days the beams were supported 5 in. from each end, loaded at midspan, and test data were obtained.

Figure B-4 shows the load-deflection curves obtained on the final cycling. Differences were within experimental error. Loads at failure were 1,280 and 1,300 lb for beams with uncoated wire and 1,380 and 1,460 lb for those with galvanized wire. Hence, the results suggested that there was no sacrifice in bond or in general beam behavior when galvanized wires were used.

Such good bonding, however, has not always been obtained with galvanized coatings. Bird (*72*) reported inconsistencies in the bond strengths between uncoated and galvanized steel in concrete. He pointed out that these inconsistencies might possibly be caused by differences in the heat treatment of the zinc-coated steel. According to Bird, if the Fe-Zn diffusion layer reaches the surface, a reaction with the wet cement paste will occur. This reaction generates hydrogen, which causes the cement matrix adjacent to the steel to become spongy. The spongy layer has a low strength, which is reflected by low values in a bond test.

TABLE B-2

TEST RESULTS, UNGALVANIZED AND GALVANIZED PRESTRESSING WIRES

ITEM	UNCOATED WIRE	GALVANIZED WIRE
Wire diameter (in.)	0.148	0.160
Ultimate tensile strength (ksi)	210	173
Modulus of elasticity (ksi)	28,100	24,000
Proportional limit (ksi)	110	140

Later work by Bird (*40*) indicated that impurities in the cement were important to the reaction. He found, for example, that hydrogen was liberated from even pure zinc unless at least 70 ppm of CrO_3 was present in the cement. This figure was later questioned by Cornet and Bresler (*71*), who found no reaction when using cements showing only 20 ppm CrO_3. These authors stated that the results reported by Bird may have been related to some exceptional quality of the cement that Bird was using.

A fairly recent report by Cornet and Bresler (*73*) compares the corrosion behavior of galvanized and bare reinforcing bars that were embedded in concrete and exposed to repeated cycles that consisted of wetting for 3 days in 4% NaCl solution, followed by drying in air for 4 days. The reinforcing bars were stressed at 20,000 psi after the concrete was cured. This treatment caused several transverse cracks to form in the concrete near the center of each specimen. The bars were 0.75-in. in diameter; the concrete specimens were 4 in. by 4 in. by 12 in. long. Corrosion was gauged by the appearance and growth of longitudinal cracks in the concrete. Figure B-5, which is a replot of data taken from Cornet and Bresler (*73*, Fig. 3), shows the differences in behavior of specimens prepared from galvanized and bare deformed bars. At least under these particular test conditions, galvanizing appreciably improved the corrosion behavior. In interpreting these data, it should be kept in mind, however, that transverse cracks were present in the specimens at the start of the alternate wetting and drying cycles and that these cracks undoubtedly were responsible for the rapid corrosion. Such cracks would not be present in prestressed concrete; hence, much longer times would be expected before the onset of corrosion under the same test conditions.

Tripler *et al.* (*74*) after an investigation of potential coating materials for reinforcing bars, recommended that zinc should be given further attention. They state that whereas zinc dissolves in both acid and alkaline solutions, the reaction rate drops to a very low value at a pH of 12.5,

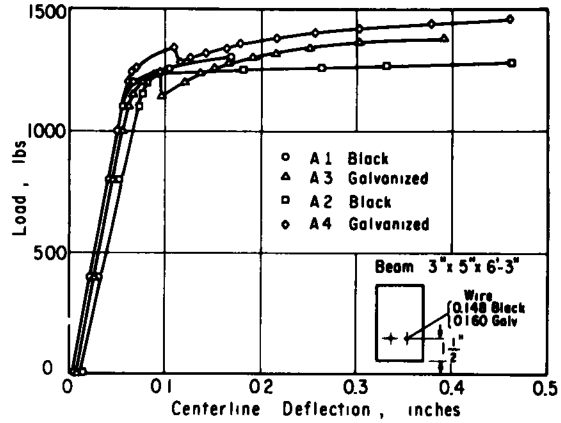

Figure B-4. Final cycle load deflections for beams stressed at 125 ksi (32).

which is the pH of the solution in the pores of the concrete. At the same time, however, they cautioned against the unqualified acceptance of galvanized coatings for use with prestressing steel because of the high stress levels employed and the resulting danger of stress corrosion. Hydrogen has been shown to be an important factor in stress corrosion, and, according to these authors, a break in the zinc coating could permit hydrogen to be generated at the cathode of the cell, which in this case would be the exposed iron. Rehm (*28*) warns against hydrogen embrittlement from this same cause. He also states that more research is needed on the corrosion of zinc in the concrete environment, even though "very favorable results have been reported by IRSID, the French research institute."

Obviously, additional information on the corrosion behavior of galvanized steel in a cement environment would be highly desirable. Tests in progress at Kure Beach, North Carolina (*75*), should, when completed, provide much valuable data. Also, helpful information should become available in the future from large-scale applications of galvanized reinforcing bars that are being installed; such as, for example, at the Civic Center at Pine Bluff, Arkansas (*76*). Woods (*70*) believes that in 1968 the usefulness of zinc coating for concrete exposed in marine environment had not been adequately demonstrated. Bird and Strauss (*77*) concluded after extensive polarization tests that zinc coatings could only postpone the corrosion of steel in a marine environment.

Lead Coatings

Another potentially useful coating for prestressing steel is lead. Tripler *et al.* (*74*) reported very low corrosion rates for lead in a simulated cement environment. Lead coatings, however, are soft, are cathodic with respect to steel, and have poor bond to the steel. Woods (*70*) states that lead is attacked by the calcium hydroxide in concrete and that in damp concrete a lead pipe could be destroyed in a few years.

Nickel Coatings

Nickel is yet another metallic coating that has a reasonably good potential. Nickel is more noble than iron and hence cannot provide sacrificial protection. Experiments by Tripler *et al.* (*74*) showed that nickel coupled to iron in a 3.5% NaCl solution saturated with $Ca(OH)_2$ would cause only a moderate electrolytic corrosion of the iron. Tests in which the nickel coating was cut with a file to expose the iron, and the specimen was then immersed in the NaCl-$Ca(OH)_2$ solution, showed little or no corrosion of the iron after 3 months' exposure.

Nickel plating has been used for dowel pins in highway construction (*78, 79*). When applied at a thickness of 0.003 to 0.005 in., the coating inhibits corrosion and thereby permits free movement of the dowels for much longer periods.

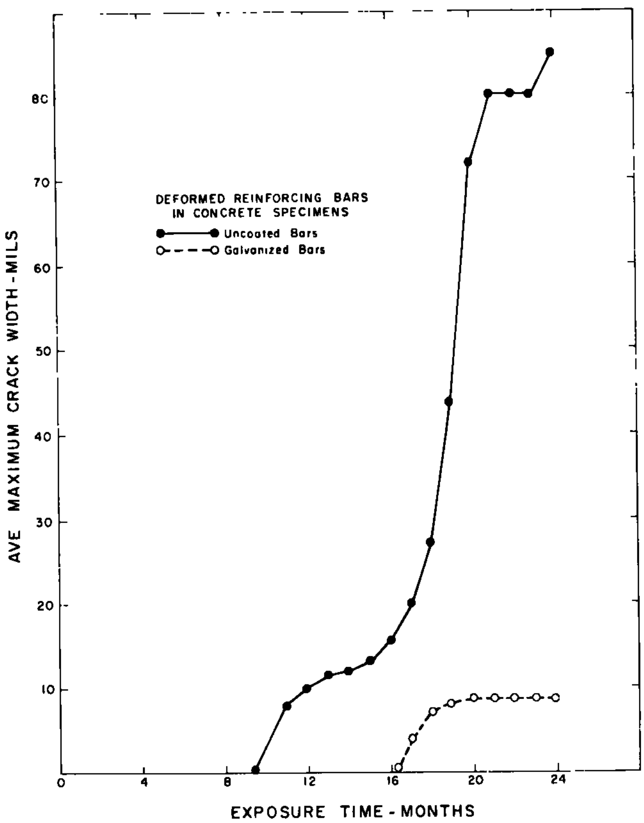

Figure B-5. Effect of exposure time (3 days, 4% NaCl solution; 4 days drying) on width of longitudinal cracks (73).

Copper Coatings

In general, copper is not corroded in concrete (*70*). Copper, however, is strongly cathodic with respect to steel, and the steel would be subject to a rapid local attack in a salt-containing concrete environment if the coating was not continuous (*77*).

Cadmium Coatings

Cadmium, like zinc, is anodic to iron. However, it is much more expensive than zinc. It has fairly good resistance to atmospheric corrosion but low resistance to corrosion by most chemicals.

A recent study by Bird and Strauss (*77*) on metallic coatings for reinforcing steel showed that cadmium provided the most satisfactory protection of any coatings that were investigated, which included zinc, nickel, cadmium, copper, lead, and tin. Both laboratory tests and long-time exposures were used. In the latter tests, zinc-plated, cadmium-plated, and uncoated steel rods were embedded in concrete and, after curing, exposed for 3 years to repeated wetting and drying using a 3% NaCl spray on alternate days. At the conclusion of this treatment, the uncoated and the zinc-coated rods showed general corrosion, whereas the cadmium-coated rods had "only blackened and, in fact, showed a slight increase in weight." The authors state that the high cost of cadmium platings could be circumvented through use of a cadmium-silver plate, the protectiveness of which was currently being investigated. They also state that when applied at a thickness of 0.001 in. a cadmium-silver coating might compare favorably in cost with other metallic coatings.

Organic Coatings

Many different organic coatings have been applied to reinforcing steel to minimize corrosion. These vary from ordinary paints to the more sophisticated resin coatings. Almost without exception the coatings have been found to reduce corrosion to varying degrees.

Practically all of the work with organic coatings as reported in the literature has been aimed at the protection of reinforcing bars rather than prestressing steel. Russian engineers have been especially active, possibly because of a current emphasis in Russia on porous concrete.

One method of producing porous concrete that is apparently used to some extent in Russia is to mix hydrogen peroxide with quicklime. The resulting reaction yields products that could accelerate the corrosion of steel, as indicated by the following:

$$H_2O_2 + CaOCl_2 \rightarrow CaCl_2 + O_2 + H_2O$$

In a study by Melenevs'ka (*80*) a ratio of H_2O_2 to $CaOCl_2$ of 1:1.5 was used, and this was added as 5% by weight of the cement to produce the porous concrete. The reported properties of the hardened concrete were:

Compressive strength	75 kg/mm² (10,650 psi)
Bulk density	1.1 g/cm³
Porosity	50%
Water absorption	23%
Air permeability	220×10^3 gm/m³-hr-mm
pH of water extract	12.4

Melenevs'ka prepared porous concrete specimens with coated and uncoated steel rods. The coatings were organic laquers applied both over bare steel and over phosphated (bonderized) rods. Tests were conducted by temperature cycling the specimens in a humidity chamber to which HCl and SO_2 vapors had been added. After 3 months, 6 months, and 12 months, specimens were removed from the chamber, broken, and the steel bars were examined. A product called "Kuzbas laquer" applied over bonderized steel gave excellent results, showing no corrosion of steel after 6 months under the test conditions. It was also found that rods coated in this way were bonded to the concrete more strongly than uncoated rods.

Solovei (*81*) subjected steam-cured concrete specimens with reinforcing bars to alternate wetting and drying cycles for periods up to 1 year. He found that protective coatings were the most reliable means of preventing corrosion. The best that was tried from the standpoint of protection and bond to the concrete was a cement-polystyrene coating. Good protection was also achieved from coatings prepared from furan resins or from a cement-poly (vinyl alcohol) mixture.

Balezin *et al.* (*82*) reported that epoxy resin coatings applied to reinforcing bars significantly increased the corrosion resistance in cyclic tests.

Tripler *et al.* (*74*) applied two proprietary coatings to reinforcing rods—one a gilsonite-bearing paint and the other an asphalt-epoxy coating. The thickness in both cases was approximately 0.010 in. Resistance and static potential measurements were made in a solution of 3.5% NaCl. The data indicated that both coatings were protective. Of the two, asphalt-epoxy gave the better results.

Epoxy-type coatings have also been considered for use with prestressing steel. According to Dumas (*83*), the only positive method of protecting prestressing wires from corrosion is to coat the wires with an epoxy-based paint of a patented composition. Tests showed that this paint did not crack under stress; also, it increased the bond between the wire and concrete. These results appear to be in keeping with statements in a recent RILEM report (*45*) to the effect that coatings based on epoxy resins adhere firmly to prestressing wire and develop "hardly any" cracks when the wires are stressed close to ultimate. It should be pointed out that organic polymers, as a class, have low elastic moduli and most of them are capable of sustaining large elongations before fracture.

Inorganic Nonmetallic Coatings

Inorganic nonmetallic coatings appear to be more suitable for protecting reinforcing bars than for protecting prestressing wires. Coatings that fall into this category include porcelain enamels, alkali silicate-bonded coatings, phosphate-bonded coatings, and portland cement-bonded coatings or slurries. Such coatings are normally quite brittle and it seems highly unlikely that one could be developed that would withstand the 0.6 to 0.7% elongation required of prestressing wires. If they could be applied after the wires were stressed, however, the cracking problem could be minimized. Actually, this approach has been used in the case of prestressed members in which a cement slurry is applied to the stressed wires prior to grouting or, in the case of pretensioned members, prior to casting of the concrete (*49, 50, 84*). The advantage of this treatment is that even though voids are formed between the wire and concrete in the casting operation, or because of subsistence, the wire is still protected by a cement layer.

Coatings for Concrete

The sealing of concrete surfaces with coatings has been receiving increasing attention during the past few years. The interest stems in part from a pressing need to find some practical method to minimize scaling and spalling of concrete bridge decks. Sealant-type coatings could be of almost equal interest in prestressed concrete structures because, if the concrete could be sealed reliably and cheaply against the entrance of moisture and other corrodents, corrosion of the prestressing steel would no longer be a problem.

Measurements by Snyder (*85*) have shown that the types of coating that have been used to minimize scaling and spalling of concrete bridge decks retard, but do not prevent, the penetration of a salt solution. Penetration data taken from Snyder's report are given in Table B-3.

The same type of absorption data were not obtained for the other coating types studied by Snyder. However, as judged from the amount of scaling observed in the freezing and thawing tests, none of these other coatings (which included epoxies, modified epoxies, latex, and many others) was especially effective in preventing absorption of the salt solution. The organic coatings studied by Snyder normally failed by pinholing, flaking, or peeling. The best performance was obtained from an epoxy coating impregnated with sand.

Smith (*86*) investigated a large number of surface coatings for the possible protection of concrete against the effects of deicing salts. None of the coatings completely sealed the surface, although all of them reduced the rate of water absorption to varying degrees. Epoxy resins were promising in laboratory tests but their field performance was disappointing. Lack of proper bond development between the epoxy and the concrete may have been partly responsible. Also, asphalt does not seal concrete effectively unless a good bond can be achieved between the two materials, and such bonding is often very difficult to obtain. Smith also points out the potential danger of coating or sealing all surfaces of the concrete. This practice prevents evaporation of water that is present in the structure at the time of coating, as well as the water that may enter later at pinholes, cracks, seams, etc. Smith states that the moisture in effect becomes trapped in the concrete and thereby permits later scaling and spalling to occur.

According to information obtained in the recent survey of state highway departments (Appendix A), two states are conducting small-scale tests of the possible effectiveness of coatings applied to prestressed girders for inhibiting the corrosion of the steel. Mississippi has applied a linseed oil coating to several bridge girders in the northern part of the state, and New Hampshire is testing an epoxy coating for the same purpose on one prestressed bridge on U. S. Route 1 near Portsmouth. So far, no test results have been reported.

A recent report of ACI Committee 515 contains an excellent review of coatings for concrete (*87*). The purpose of the report is to guide users in the selection of a coating to prevent chemical attack of the concrete in various corrosive environments.

In summary, it would appear that the general concept of completely preventing corrosion of prestressing steel by encasing the prestressed member in a watertight sheath of coating material is possible in theory but that it is an impractical approach. It would be extremely difficult to permanently prevent moisture, corrodents, and oxygen from entering the structure through any breaks in the coating that might develop. Once this happened, the coated member would tend to stay permanently moist, because evaporation from the surface could then occur only at an extremely slow rate. The presence of permanent wetness could, under some conditions, cause an accelerated corrosion of the prestressing steel.

CATHODIC PROTECTION

Cathodic protection is a widely used technique for minimizing steel corrosion in structures exposed to aggressive environments. Briefly stated, the method consists of applying a direct current to the steel of such polarity and intensity as to raise the electrical potential of the cathodic areas to that of the anodes. When this is accomplished successfully, corrosion currents can no longer flow and corrosion cannot proceed.

The current that is needed for this type of protection can be obtained from a rectifier or it can originate from electrically connected sacrificial anodes. In either case, an electric cell is formed in which the impressed current flows in a direction opposite to the corrosion current. The required currents are normally quite small. For reinforced concrete, current densities of the order of 100 ma per square foot of steel surface are usually sufficient (*56*). Applied voltages are never higher than about 1 or 2 volts.

Although the principal applications of cathodic protection are for buried pipelines and storage tanks, Stratfull (*19*) has applied the technique to reinforced concrete beams and deck units of a bridge over San Francisco Bay that was deteriorating because of corrosion of reinforcing bars (San Mateo-Hayward Bridge). Electrical connection to the concrete was achieved by carbon anodes sealed into the structure at strategic locations with a slurry consisting of one part by volume raw gypsum, four parts steam-exploded mica, and one-half part $CaCl_2$. After 1 year the system seemed to be working effectively.

Whenever complete cathodic protection is in effect, hydrogen gas is liberated at the steel surface according to the reaction:

$$H_2O + e \rightarrow \tfrac{1}{2} H_2 + OH^-$$

In addition to the hydrogen gas formation, the electrolyte surrounding the steel becomes more and more basic as the current flow continues.

Thalmann (*88*) investigated the effects of calcium chloride additions to concrete on the potential and current values that might be needed for cathodic protection. Current discharged from steel encased in concrete and maintained at a potential of —0.27 volt (SCE) was 0.16 ma/ft² with no chloride addition and 850 ma/ft² for concrete with a 6% $CaCl_2$ addition. With 2% $CaCl_2$ in the concrete, the current was 48 ma/ft². The current collected by the protected steel specimen at —0.85 v (SCE) was 0.32 ma/ft² for concrete with no chloride and 0.80 and 2.20 for additions of 2% and 6% $CaCl_2$.

Franquin (*89*) has investigated loss of bond between steel

TABLE B-3

PENETRATION DATA

COATING APPLIED TO CONCRETE	AMOUNT OF 3% NACL SOLUTION ABSORBED IN POROUS CONCRETE AFTER IMMERSION FOR INDICATED NO. OF DAYS				
	1	4	8	12	50
None	10.7	10.9	11.2	11.4	11.4
Linseed oil [a]	3.0	4.4	5.8	6.5	9.2
Silicone resin	0.8	4.4	10.3	10.7	11.5

[a] 50 parts by volume boiled linseed oil and 50 parts kerosene.

and concrete that might result from cathodic overprotection. His results show that for current densities lower than 1 amp/m^2 of steel area there is apparently no loss of bond. These current values are considerably higher than those required for cathodic protection (generally 1 to 1.5 ma/m^2 are employed) but he cautions against situations which would result in poor current distributions and high local current densities.

In studies reported by Scott (*38*), cathodic tests were conducted using tapered steel electrodes cast in concrete that were spring-loaded so as to accelerate bond failure. Under an applied voltage of 2.14 v (SCE) and with an initial current density of approximately 400 ma/ft^2 one specimen failed after 197 days. Other specimens tested at lower voltages showed bond strengths comparable to the control specimens after 880 days and 994 days. The specimens were exposed to a 1% NaCl solution. It was concluded that the bond between the concrete and the steel would not be impaired at any polarization potential less cathodic than about −1.00 v (SCE).

Unz (*90*), in discussing the destruction of the bond between the concrete and the steel at cathodic zones, pointed out that the disintegration of the bond requires Na or K ions and that normal portland cement does not contain a sufficient quantity of these ions to cause problems. However, in soils with excessive salt contents, a steady migration of the Na and K ions to the cathode occurs. This migration could also occur for a prestressed concrete structure under cathodic protection and exposed to a marine environment.

In way of summary, it should be pointed out that cathodic protection of bridge superstructures presents certain problems not encountered in concrete structures buried in the ground. In the case of buried structures, not only is the moisture content of the concrete more uniform, but anodes can be placed in the ground adjacent to the structure rather than in the concrete itself. With exposed bridge girders, large changes occur in moisture content which cause an accompanying change in resistivity. In addition, variations in moisture content are often present along the length of the girder. According to Woods (*70*) it is extremely difficult under such conditions to ensure that all parts of the reinforcing steel are receiving the correct amount of impressed current and, conversely, that none of the steel areas is receiving such a large amount as to create problems from hydrogen evolution. The consensus seems to be that cathodic protection, although a valuable technique for the corrosion engineer, is not particularly well suited for the protection of above-grade reinforced concrete structures (*70, 37, 91, 92*).

CORROSION PROTECTION OF END ANCHORAGES

Corrosion protection of anchorages in post-tensioned girders has been the cause of some concern because, if bonding of the tendon to the grout is substandard, corrosion failures at the anchorages could cause loss of prestress. Also, staining of the concrete by corrosion of the anchorage could be unsightly.

Pretensioned girders do not, of course, require anchorages. Nevertheless, because the pretensioning steel is exposed at the girder ends, corrosion can occur and, as in the case with the post-tensioning, cause unsightly staining of the concrete.

There is only one systematic investigation of end protection systems for post-tensioned beams reported in the published literature. This was carried out by Roshore (*93*). Twelve types of end protection systems were used on 20 test beams, each of which was 10 in. by 16 in. in cross section by 8 ft long. In the case of flush anchorages, the protective layer filled the recess in the ends of the beam, whereas for external anchorages the protective layer formed a capping block on the rectangular cross section.

The protective systems, which in each case provided a 1.5-in. cover for the steel, consisted of air-entrained concrete, epoxy concrete (3 parts epoxy binder, 7 sand, and 10 aggregate), and a sand-cement mortar with an admixture of aluminum powder.

Testing was done at Treat Island, Maine, at a tidal location where the test beams were exposed to twice-daily inundations of sea water. After an exposure of 2 years, in which the beams were subjected to 195 cycles of freezing and thawing, the anchorages were inspected. The observations made at this inspection were that (1) the epoxy concrete was superior to portland cement concrete with respect to bond separation at the beam interface; (2) sandblasting or bush-hammering the ends of the beam before application of the portland cement concrete layer did not eliminate bond separations for the beams with external anchorages; and (3) layers of portland cement concrete gave better protection on recessed anchorages than on external anchorages.

Thus, the two most important findings of this investigation are:

1. That epoxy concrete is superior to portland cement concrete.
2. That recessed anchorages can be protected against corrosion with better effectiveness than external anchorages.

In the case of pretensioned beams, protection of the exposed prestressing strand at the ends is not believed critical since (because of penetration of neat cement into the strand interstices during concrete compaction) corrosive solutions are not free to travel along the center wire for any appreciable distance (see Appendices F and H). Protection is needed, however, to avoid unsightly rust staining. Epoxy cements dabbed over the steel and adjacent concrete have been found to be reasonably satisfactory for this purpose.

SUMMARY OF LITERATURE REVIEW

A highly condensed summary of the current state of knowledge with respect to the corrosion of prestressing steel in concrete follows:

1. The protective effect of concrete on steel stems from its high alkalinity, which passivates the steel surface. Lowering the alkalinity, such as by carbonation or by influx of foreign ions, tends to destroy this passivity.
2. The only cases of serious corrosion of steel in concrete have been those in which chlorides or other corrosive ions were present in the environment.
3. Steel corrosion in the presence of chlorides is an

electro-chemical process, with the moist chloride-containing concrete serving as the electrolyte.

4. Chlorides in concrete have a deteriorating effect on the passivated film on the steel, thereby making the steel more susceptible to corrosion.

5. Chloride ions react with hardened cement to form the relatively insoluble calcium chloroaluminate. The reaction does not proceed to completion, however, because appreciable amounts of soluble chloride are still present in hardened cement pastes that contain as little as 0.25 weight percent $CaCl_2$.

6. Poor agreement exists among investigators as to a threshold level of chloride in concrete below which no corrosion of steel will occur.

7. Poor agreement also exists as to the minimum concrete cover needed for steel protection. This poor agreement is caused partly by a lack of data on permeability of the concrete covers and partly by differences in exposure conditions.

8. Contradictory evidence is present in the literature as to the effect of cracks on steel corrosion, and more especially as to the effect of crack width.

9. Various types of coatings have been used for protecting steel from corrosion. The two most common types are hot-dipped galvanized and epoxies.

10. Cathodic protection of steel in concrete has been used with a limited success on one bridge structure.

11. The best corrosion protection of end anchorages in post-tensioned girder has been achieved with an epoxy concrete. Portland cement concrete layers give better protection on recessed anchorages than on exterior anchorages.

12. Up to the present, no catastrophic failures have been reported for prestressed concrete bridges. Furthermore, for those rare cases where serious corrosion of prestressing steel has occurred, the failures are explainable, and the corrosion could have been avoided either by proper use of materials or through greater attention to accepted construction practices.

APPENDIX C

STUDIES OF BRITTLE FRACTURE OF PRESTRESSING STEEL

The possibility of brittle fracture of tendons in prestressed concrete structures is a cause of grave concern to many bridge engineers and designers. As pointed out in Appendix B, high-strength steels are particularly susceptible to this phenomena, especially when used at high stress levels.

Two mechanisms are capable of causing brittle failures of prestressing steel. The first is stress-corrosion cracking (SCC) and the second is hydrogen-embrittlement. Fortunately, fracture of the steel by either of these mechanisms occurs only during exposure to certain specific environments. The goal of the research as described in this appendix was therefore to determine if any of the environments that might logically be encountered in bridge superstructures were capable of causing brittle fracture of prestressing steel.

TEST EQUIPMENT AND TEST PROCEDURES

Figure C-1, a partial view of the testing rack, shows 5 of the 16 test stations. As shown in the figure, Lucite cylinders for containing the test solution were sealed around each specimen during testing.

Most of the specimens were prepared from cold-drawn wire, 0.106-in. diameter, obtained from the CF&I Steel Corp. The reported chemical analysis was C 0.70, Mn 0.94, P 0.015, S 0.016, and Si 0.20. The ultimate tensile strength was 286,000 psi.

The as-received cold-drawn wire had not been stress relieved. Some specimens were tested in this condition, whereas others were tested after stress relieving in the laboratory. The same temperatures and times as those used by the manufacturer were employed for these laboratory stress-relieving operations.

All specimens were 13 in. long, with buttonheads at each end. Figure C-2 shows the type of grip that was used.

Stressing of specimens was done with the hydraulic loading jack shown in Figure C-1. After stressing to the desired level, shims of appropriate thickness were inserted between the stressing collar and the support frame so as to maintain a constant elongation throughout the testing.

Specimen failures in some cases were recorded by a time clock activated by a cut-off switch placed below the bottom grip. In most cases, however, times for failures, when they occurred, were obtained from visual observations.

STUDIES OF POSSIBLE STRESS-CORROSION CRACKING IN CHLORIDE ENVIRONMENTS

Contradictions exist in the literature as to whether prestressing steel is susceptible to stress-corrosion cracking in chloride environments. Gilcrist (*48*), for example, has reported that such cracking does occur. Rehm (*28*), on the other hand, has stated that his experience did not confirm that chlorides could cause the SCC type of failure.

Because chlorides are often present in prestressed bridge

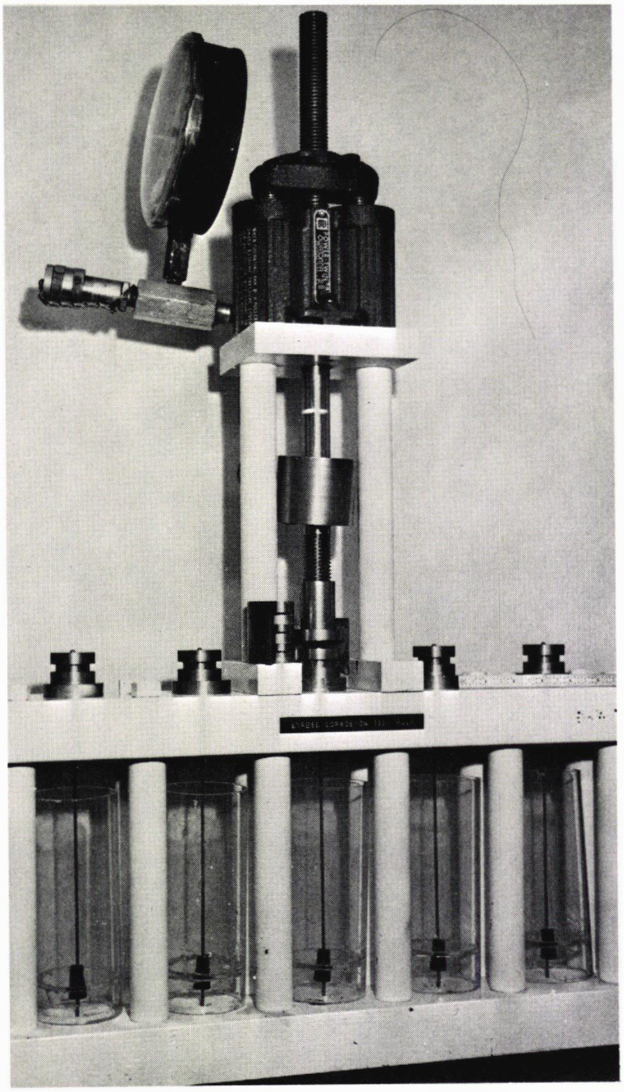

Figure C-1. Partial view of test rack for studying brittle fracture. Hydraulic loading jack on middle specimen was used for stressing.

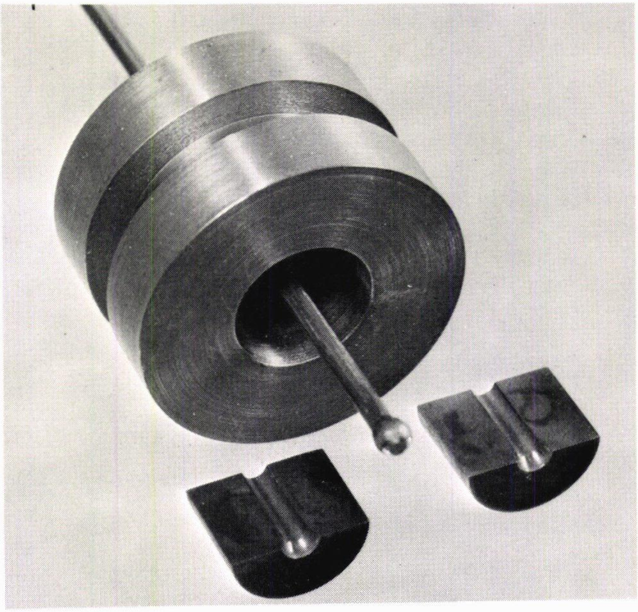

Figure C-2. Type of grip used for wire specimens.

environments (Appendices A and B), and because stress-corrosion cracking of tendons could be quite dangerous to bridge safety, numerous tests were conducted in an attempt to resolve this SCC question with respect to chlorides.

The first testing was done at room temperature. Most of these tests were confined to 3.5% by weight solutions of NaCl and $CaCl_2$, using both as-received and stress-relieved wires. A pitting-type corrosion of the steel occurred in all tests, but in no case did any of the specimens fail by a brittle fracture, regardless of stress level. The same results were observed in simulated concrete environments; i.e., environments containing saturated $Ca(OH)_2$ with 3.5% of either NaCl or $CaCl_2$.

Because quenched and tempered steel is known to be more susceptible than cold-drawn steel to brittle fracture in nitrate environments, stress-corrosion tests were made of quenched and tempered wires in an environment containing chlorides. The quenched and tempered specimens were prepared by first heating the as-received, cold-drawn wire to 1550°F for 1 hr, then quenching in oil, and finally tempering for one hour at 600°F. The specimens after quenching and tempering were tested in 3.5% NaCl solutions while stressed at the same levels used for the cold-drawn wires. No brittle fractures were observed up to the maximum testing time of 800 hr.

Tests were also conducted in 3.5% NaCl and $CaCl_2$ solutions at 200°F for 340 hr. For these tests, small stressing frames were placed inside a constant-temperature chamber. A condenser was attached to each frame to prevent evaporation of the solutions. Again, even under these highly accelerated test conditions, no brittle fractures were observed at any of the stress levels (175, 200, and 225 ksi). Instead, a general corrosion occurred, accompanied by a localized pitting at various locations along the test area.

The conclusion reached at the termination of this testing was that stress-corrosion cracking of prestressing steel *will not* occur in the type of chloride environments likely to be encountered in prestressed bridge girders.

HYDROGEN EMBRITTLEMENT CRACKING

Hydrogen embrittlement cracking of steel under stress can occur whenever hydrogen enters the steel structure. The entrance of the hydrogen usually occurs by cathodic charging, and such charging is possible whenever the steel is electrically coupled in a corrosive environment to a more anodic metal (for example, zinc or aluminum). Because both of these metals might, on occasion, be present in prestressed concrete structures, and because in some cases failures have been ascribed to their presence (Appendix B), various experiments were performed to determine if failures of this type could be obtained under laboratory test conditions.

The first hydrogen embrittlement tests were conducted with specimens that had been galvanized by hot dipping for 1 min at 840°F. A longitudinal strip of the galvanized layer was removed from the test section on some of the specimens to give an iron-zinc couple; on others, the coating was left intact. The specimens were then stressed to various levels and exposed to corrosive solutions that might cause cathodic charging of the steel. As given in Table C-1, no hydrogen embrittlement failures occurred under any of the test conditions. This same type of result was obtained with 0.25-in. galvanized prestressing wires with a strip of coating removed that were stressed at 60% of ultimate and then tested for 4,300 hr in a cement-mortar environment. Even though an aerated 3.5% NaCl solution was in contact with the mortar throughout this testing, no failures occurred (Appendix E).

The possible effect of aluminum-steel couples was investigated in a second type of experiment. For these tests, aluminum wires, ⅛-in. diameter, were wrapped tightly around prestressing wire specimens for the full length of the test section. The results are given in Table C-1. Although the aluminum was severely attacked, none of the steel wires failed by hydrogen embrittlement cracking under any of the highly accelerated test conditions that were used.

Because no brittle fractures were observed in any of the hydrogen embrittlement testing, a conclusion that might be reached is that hydrogen embrittlement failures are not a cause of concern when zinc or aluminum is coupled

TABLE C-1
SUMMARY OF HYDROGEN EMBRITTLEMENT TESTS

SPECIMEN DESCRIPTION	INITIAL STRESS (ksi)	ENVIRONMENT[a]	TESTING TIME (hrs.)	TEST RESULTS
Galv	225	Sat Ca(OH)$_2$	784	No failures Some loss of zinc
	200			
	175			
	150			
Galv	225	Sat Ca(OH)$_2$ + 3-1/2% NaCl + O$_2$	720	No failures More zinc loss than for exposure in sat Ca(OH)$_2$
	200			
	175			
	150			
	125			
	100			
Galv , 1/8' wide strip removed	225	Sat Ca(OH)$_2$ + 3-1/2% NaCl	336	No failures Some loss of zinc
	200			
	175			
	150			
	125			
	100			
Galv , 1/16" strip removed	200	Sat NaOH	790	No failure Some loss of zinc
Galv , 1" strip removed	225	Sat. NaOH	790	No failure Some loss of zinc
Galv , ctg notched	100	Sat NaOH	790	No failure Some loss of zinc
Al wire around as-rec'd steel	200	Sat Ca(OH)$_2$ only	360	No failure Severe attack of Al wire
	175	Ca(OH)$_2$ + 2% H$_2$S	360	
	175	Ca(OH)$_2$ + 9% H$_2$S	360	
	175	Ca(OH)$_2$ + 17% H$_2$S	360	
	175	Ca(OH)$_2$ + 28% H$_2$S	360	
As-rec'd wire	200	Ca(OH)$_2$ + 2% H$_2$S	360	No failure
Al wire around as-rec'd steel	200	Sat NaOH + 2% H$_2$S	500	No failures Al wire completely dissolved
	225	Sat NaOH + 9% H$_2$S	500	
	225	Sat NaOH + 17% H$_2$S	500	

---

[a] NaCl added as wt percent, H$_2$S solution added as percent by vol Percentage H$_2$S in last three series of tests refers to amount of a saturated H$_2$S solution added Probable reason why presence of H$_2$S did not cause brittle cracking was precipitation of CdS The pH of solutions at test termination was 13 2

electrically to the steel in prestressed concrete structures. The fact remains, however, that such failures have been reported in the field (*61*). In this connection, the presence of "poisons" in stimulating the entrance of hydrogen into the steel cannot be overlooked. Sulfide and arsenide ions are the most common poisons, and, whenever these ions are present, considerable amounts of hydrogen can enter the steel. Sulfides present in some cements could conceivably furnish the necessary "poison." The presence or absence of sulfide, or in the type of sulfide present, might possibly explain the discrepancy in results between various investigators.

HYDROGEN SULFIDE CRACKING

Hydrogen sulfide environments are known to cause brittle cracking of steels that are in a highly stressed condition. Therefore, because this gas could conceivably be present in small amounts at some bridge locations, and also because H_2S might be generated when sulfides are present in a cement or aggregate, a rather extensive study was conducted of H_2S cracking of prestressing steel.

TABLE C-2

OCCURRENCES OF FAILURE

INITIAL STRESS (KSI)	FAILURE TIMES OF AS-RECEIVED WIRES IN H_2S ENVIRONMENT (HR)	
	DISTILLED WATER	SAT. $CA(OH)_2$
100	1.42	>8, <24
125	1.33	7.98
150	1.25	4.90
175	1.17	6.93
200	1.25	7.07
225	1.25	2.25

In a first series of tests, as-received wires were stressed at initial levels of 100 to 225 ksi and then were exposed to distilled water through which H_2S was bubbled. All wires failed in from 1 to 2 hr by brittle fracture. However, no correlation was observed between failure time and the initial stress level. Similar tests with wires that had been stress relieved yielded failure times of from 1.5 to 7.5 hr, but again there was no correlation between time for failure and the initial stress level. Polishing of as-received wires by either hand polishing with No. 600 abrasive or by electrolytic polishing was found to have no significant effect on the failure times.

Galvanizing of wires for 1 min at 840°F prevented failure up to a maximum testing time of 10 hr. However, when narrow strips of the zinc coating were removed prior to testing, failures occurred in approximately the same time intervals observed for the uncoated stress-relieved specimens.

When saturated $Ca(OH)_2$ solutions were substituted for the distilled water, failures occurred after somewhat longer times (Table C-2).

The pH of the solutions decreased from 6.1 to about 5.0 during the testing in distilled water and from 12.5 to about 8.0 in saturated $Ca(OH)_2$.

Tests in the distilled water-H_2S environment were also made with as-received wire specimens protected with various thicknesses of a sand-cement mortar (sand-cement ratio 3:1, w/c 0.45, Type I cement). With a ⅛-in. cover, failures occurred within 4 hr. However, with the heavier covers (¼, ⅜, ½, and ¾ in.) no failures were observed up to the maximum testing time of 150 hr. Figure C-3 shows the specimens in the stressing frame prior to testing.

The effect of H_2S concentration was studied by bubbling mixtures of H_2S and nitrogen through the solutions. Fig-

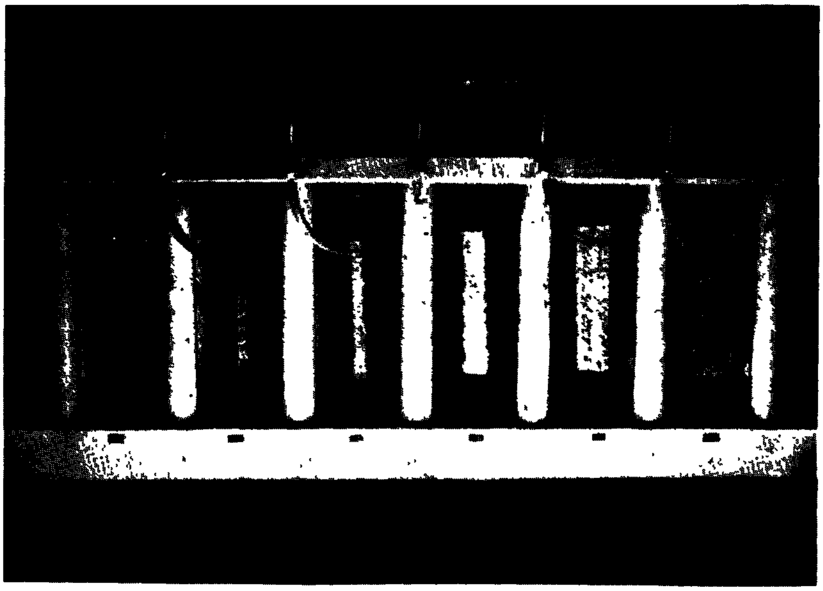

Figure C-3. Arrangement for testing effect of concrete cover on H_2S cracking.

ure C-4 shows that the time for failure of as-received wires in distilled water increased with decreasing concentrations of H_2S. The same pattern was observed when saturated $Ca(OH)_2$ was substituted for the distilled water, except that in this case all times for fracture were increased by a factor of about five.

During the course of the testing program on H_2S fracture it was discovered that the wire received from the steel mill contained very small longitudinal cracks. Although the presence of these cracks may have affected the times for failure, it is highly unlikely that the inherent susceptibility of the steel to H_2S cracking was affected by their presence.

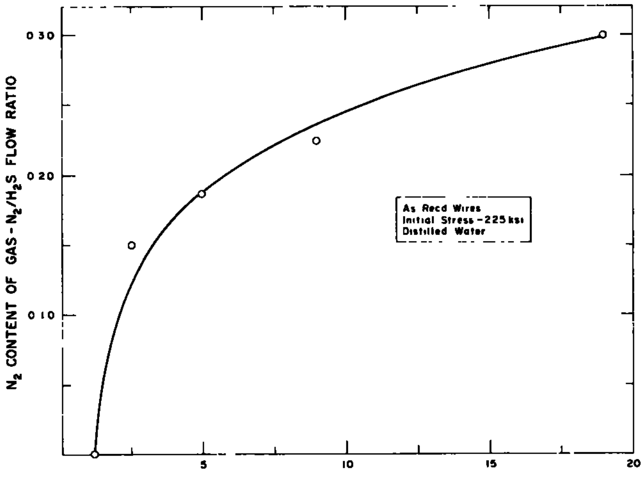

Figure C-4. Effect of H_2S dilution by nitrogen on fracture time.

SUMMARY OF BRITTLE FRACTURE TESTS

The tests on brittle fracture of prestressing steel can be summarized as follows:

1. Stress-corrosion cracking of the steel did not occur in chloride environments, even under the most highly accelerated test conditions that were used.
2. No hydrogen embrittlement failures were observed when steel specimens were coupled to either zinc or aluminum and exposed to simulated concrete environments both with and without chlorides.
3. Cracking of prestressing steel occurred after only short exposure times in distilled water into which H_2S was bubbled, and in somewhat longer times when a saturated $Ca(OH)_2$ solution was substituted for the water. Other findings in the H_2S fracture testing were:
    a. Galvanizing of the wire prevented brittle failures of the steel, but only if the coating was continuous.
    b. Sand-cement mortar, when applied at thicknesses of 0.25 in. or greater, prevented brittle fractures up to a maximum testing time of 150 hr.
    c. The times for fracture increased with decreasing H_2S content of the gas bubbled through the solutions.

APPENDIX D

STUDIES OF ELECTROCHEMICAL CELL ACTION

To take proper measures to prevent the corrosion of prestressing steel in concrete, it is important to learn as much as possible about the mechanisms responsible for the corrosion. Because the corrosion is known to be caused by electrochemical cell action, studies of the types of cells that are active, and the factors that affect the corrosion rate of steel in such cells, could be highly rewarding in indicating directions that might be taken to prevent future corrosion problems.

As indicated in Appendix B, earlier investigations have established that steel does not corrode when embedded in concrete unless foreign ions (of which chloride is the most common) are present. Therefore, almost all of the research to be described in this appendix was conducted in chloride-bearing environments. Also, because several earlier investigations have pointed to the possible importance of oxygen-gradient cells, much of the emphasis in the research was placed on cells of this type.

Prestressing steel, 0.106 in. in diameter, obtained from the CF&I Steel Corp. was used in all of the tests that are described in the following sections.

STUDIES WITH OXYGEN-GRADIENT CELLS

In a preliminary series of tests, as-received wires stressed at various levels from 100,000 to 225,000 psi were exposed to the following solutions for 800 hr:

1. Saturated $Ca(OH)_2$.
2. Saturated $Ca(OH)_2$ + 3.5% NaCl.
3. Saturated $Ca(OH)_2$ + 3.5% NaCl + O_2 (bubbled).

The only corrosion observed after the testing was at the solution-air interface in solutions 1 and 2. Also, when this interface was eliminated by casting a layer of cement mortar around the top of the wires, no corrosion occurred. Thus, the results of these early tests tended to confirm the importance of both oxygen gradients and chloride ions to electrochemical cell action.

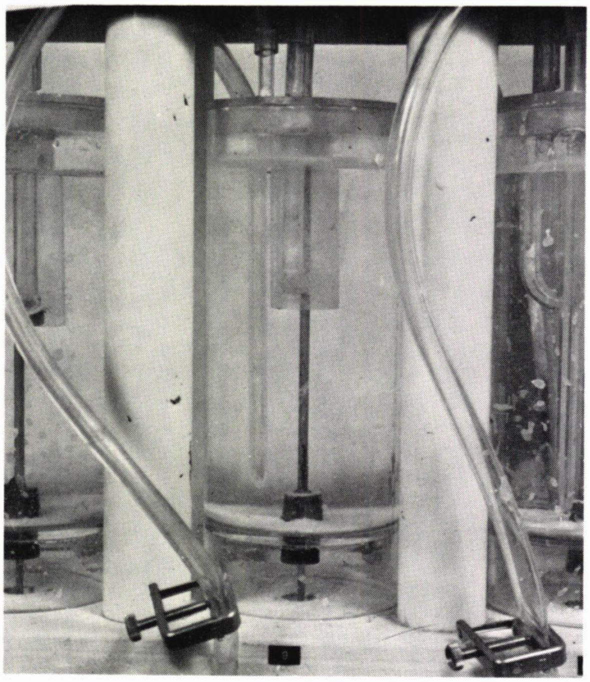

Figure D-1. First cell type used for testing effects of oxygen concentration gradients. Oxygen was introduced at bottom of cell; nitrogen into Lucite chamber at top. Specimen was stressed.

To more clearly define the effects of oxygen concentration gradients on steel corrosion, a special cell (Fig. D-1) was designed and constructed. After stressing of the wire (which in this case had no protective mortar layer at the solution-air interface) a solution of saturated $Ca(OH)_2$ + 3% NaCl was placed in the large Lucite tube. Oxygen was then allowed to bubble into the bottom of the large tube and nitrogen was allowed to bubble into the small tube at the top of the cell. Corrosion of wires tested in this manner occurred at the bottom of the test cell at the crevice where the wire was in contact with the rubber stopper. An oxygen deficiency would be expected at this location, thereby establishing the necessary conditions for a differential oxygen concentration cell.

A second type of test was made using the cell shown in Figure D-2. In this case, nitrogen was introduced into the center tube surrounding the specimen, which was unstressed, and oxygen was introduced into both ends of the larger tube. Corrosion of the wire in this particular cell was most pronounced in the inner tube where the oxygen concentration was lowest.

Tests were also made with various other cell configurations and under various exposure conditions. The results, which are summarized in Table D-1, show that oxygen concentration cells are highly important to the corrosion of steel in simulated concrete environments that contain chlorides.

It is important to point out that although many of the wires were highly stressed in the tests in Table D-1, not one failure occurred by brittle fracture. Instead, the failures occurred by reduction of wire cross section which, in turn,

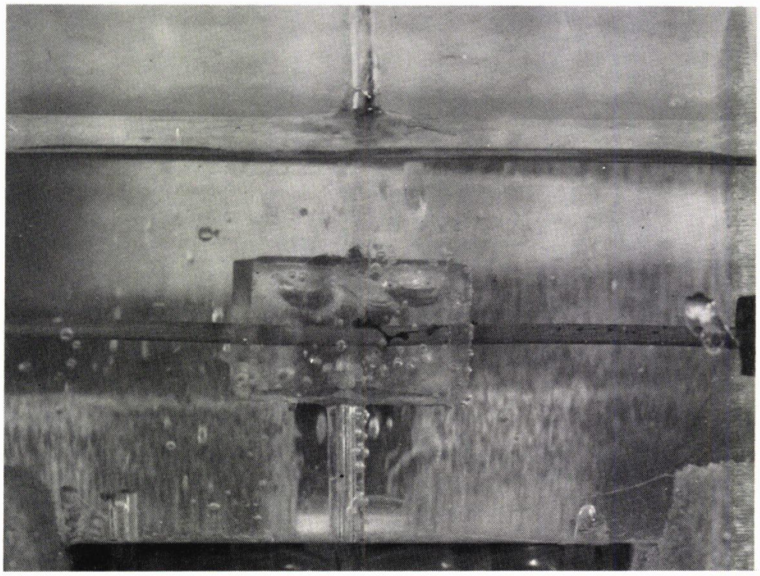

Figure D-2. Second cell type used for testing effects of oxygen gradients. Nitrogen was introduced into inner Lucite chamber; oxygen at each end of larger tube. Specimen was unstressed.

was caused by a localized pitting of the steel. Under some of the test conditions failures of this type occurred in as short a time as 10 days.

ANODIC POLARIZATION STUDIES

Anodic polarization curves were obtained from measurements made with a Wenking potentiostat. This instrument has the very desirable feature of maintaining a constant potential at any given voltage setting, irrespective of the current flow through the cell.

One of the several purposes of the potentiostatic measurements was to determine the amount of chloride ion that was necessary to destroy passivity of the steel in simulated concrete environments. Figure D-3 shows the curves obtained when varying amounts of NaCl were added to a saturated $Ca(OH)_2$ solution into which oxygen gas was bubbled. With curves of this type, passivation of the steel is indicated whenever a sharp break occurs (i.e., when the potential on the cell can be increased by a significant amount without an accompanying increase in the current flow). As shown in Figure D-3, a strong passivation occurred when no chloride was present; however, the passive state was markedly reduced when the sodium chloride additions were 0.2% by weight or greater. In fact, the passivation had almost completely disappeared when the NaCl content reached only 0.5%.

Figure D-4 shows similar curves when the solutions were de-aerated by allowing nitrogen to bubble through the cells during measurement. Note that there is little change in pattern over the curves shown in Figure D-3. Thus, the results with both types of solution demonstrated that the passivation of prestressing steel is markedly reduced by as little as 0.2% weight percentage of NaCl, and, further, that the passivation is virtually destroyed by the presence of as little as 0.5%.

The polarization studies showed that the chloride ion is very effective in causing breakdown of passivation of steel in simulated concrete environments. However, even though passivation is destroyed in both oxygenated and deoxygenated solutions, no corrosion of the steel would occur in the deoxygenated case.

To confirm that corrosion of the steel is possible at low concentrations of NaCl under the conditions of an oxygen-concentration cell, steel wires were exposed in cells of this type to the same solution concentrations as were used for the potentiostatic measurements. In these tests, corrosion of the steel was found to occur at concentrations as low as 0.4% NaCl. Thus, these corrosion results are in agreement with the anodic polarization curves, inasmuch as the polarization data showed that loss of passivation of the steel is pronounced at 0.5% NaCl, whereas the corrosion testing demonstrated that steel corrosion will occur at approximately this same chloride concentration. The corrosion will occur, however, only if oxygen concentration gradients are present.

For the polarization measurements plotted in Figures D-3 and D-4, the NaCl was added to the $Ca(OH)_2$ solutions before the wires were placed in the cells, and for this reason there was no opportunity for a passivating film to form on the steel surface prior to exposure to chloride ions. To determine if different results might be obtained if the wires were passivated prior to exposure, an additional set of specimens was first immersed in chloride-free saturated $Ca(OH)_2 + O_2$ for a period of 24 hr to permit passivation to occur. Additions of NaCl were made to the solutions in various amounts. Corrosion was observed to be starting on steel wires when 0.5 to 0.6% had been added. Thus, the same corrosion pattern was observed as with the specimens that had not been passivated prior to test. In view of these results it can be concluded that the presence of a passivated film on prestressing steel does not affect the corrosion resistance once chloride ions have diffused through the concrete and reached the steel surface.

EFFECT OF INHIBITORS

The possibility of using inhibitors in concrete to prevent corrosive attack caused by the presence of chloride ions has been considered by a number of investigators. Sodium nitrite was found to be promising as an inhibitor in an investigation conducted by Gouda and Monfore (*94*); hence, sodium nitrite was selected for investigation in the present study.

Figure D-5 shows the anodic polarization curves that were obtained when various amounts by weight of the sodium nitrite inhibitor were added to saturated $Ca(OH)_2 +$ 3.5% NaCl solutions. Figure D-5 shows that $NaNO_2$ in amounts as low as 2% permits a partial steel passivation to occur.

Figure D-6 shows the effect of a 4% addition of $NaNO_2$ on saturated $Ca(OH)_2$ solutions with concentrations of NaCl of 1% and lower. Comparison of these curves with those of Figure D-3 shows clearly that the $NaNO_2$ tends to restore some of the passivation that is lost when chlorides are present in the solutions.

That sizeable amounts of $NaNO_2$ do, in fact, act as an inhibitor and prevent the steel from corroding in a simulated concrete environment was demonstrated by a final series of tests in which unstressed wires were exposed in oxygen-gradient cells to saturated $Ca(OH)_2 + 3.5\%$ NaCl solutions containing three different amounts of $NaNO_2$. Corrosion of the steel was observed within a few hours when the $NaNO_2$ contents were either 0.2 or 0.5%; however, when 8.0% was present, no steel corrosion was observed up to the maximum testing time of 7 days.

SUMMARY OF STUDIES OF ELECTROCHEMICAL CELL ACTION

The studies with electrochemical cells in simulated concrete environments have shown that oxygen concentration gradients are highly important to the corrosion of steel in concrete. Cells of this type could be produced in prestressed concrete from (1) cracks in the concrete that reach the steel, (2) differences in permeability of the concrete to air at various locations along the length of a girder, or (3) air-containing voids at the interface between steel and concrete or between steel and grout. For such cells to be active to the point of being dangerous, however, both moisture and

TABLE D-1

SUMMARY OF TESTS WITH OXYGEN-GRADIENT CELLS

DESCRIPTION OF TEST	TEST ENVIRONMENT	RESULTS
Wire stressed to 100 ksi N_2 bubbled around top of wire. O_2 bubbled into bottom of cell.	Sat. $Ca(OH)_2$ + 3.5% NaCl with O_2 and N_2	Corr. at seam between lower stopper and wire. Fractured after ten days.
Wires stressed to 150, 175, 200 and 225 ksi. N_2 bubbled around top of wire. O_2 bubbled into bottom of cell.	Sat. $Ca(OH)_2$ + 3.5% NaCl with O_2 and N_2	Removed after 980 hours. Wires corroded at bottom. One failure.
Unstressed wire placed horizontally in cell N_2 bubbled around center portion of wire. O_2 bubbled over both ends of wire.	Sat. $Ca(OH)_2$ + 3.5% NaCl with O_2 and N_2	Corrosion occurred along center portion of wire. Exposure time - 35 days.
Wires stressed to 100, 125, 150, 175, 200 and 225 ksi. One-half of volume (vertical plane) of the cell was occupied by styrafoam. O_2 bubbled into bottom of cell.	Sat. $Ca(OH)_2$ + 3.5% NaCl with O_2	Corrosion occurred along center portion of wire closest to styrafoam. No wire failures. Exposure time - 9 days.
Wires stressed to 100, 125, 150, 175, 200 and 225 ksi. Center portion of wire surrounded by ring of styrafoam. O_2 bubbled into bottom of cell.	Sat. $Ca(OH)_2$ + 3.5% NaCl with O_2	Central portion corroded. No wire failures. Exposure time - 10 days.
Unstressed wire in vertical position. N_2 bubbled around wire in center portion. O_2 bubbled into cell at top and bottom.	Sat. $Ca(OH)_2$ + 3.5% NaCl with O_2 and N_2	Corrosion along entire length of wire. Exposure time - 13 days.
Unstressed wire in vertical position. Top of wire encased in cement mortar. Bottom of wire surrounded by closed-end tube. O_2 bubbled into bottom of cell.	Sat. $Ca(OH)_2$ with O_2	No corrosion. Exposure time - 13 days.
Same as above test.	Sat. $Ca(OH)_2$ + 3.5% NaCl + O_2	No corrosion Exposure time - 13 days.
Sample stressed to 100 ksi, oxygen bubbled into bottom of cell	Sat. $Ca(OH)_2$ + 0.4% NaCl with O_2	Corrosion at bottom. Exposure time - 10 days. No failure.
Sample stressed to 125 ksi, oxygen bubbled into bottom of cell.	Sat. $Ca(OH)_2$ + 0.5% NaCl with O_2	Corrosion at bottom Exposure time - 10 days. No failure
Sample stressed to 150 ksi, oxygen bubbled into bottom of cell.	Sat. $Ca(OH)_2$ + 0.6% NaCl with O_2	Same as above.
Unstressed wire in vertical position. O_2 bubbled around top of wire. Bottom of wire encased in cement mortar where it entered cell.	Sat. $Ca(OH)_2$ + 3.5% NaCl with O_2	Corrosion occurred below O_2 inlet within few hours. Exposure time - 7 days.
Same as for above specimen except top of wire was below solution-air interface.	Sat. $Ca(OH)_2$ + 0.6% NaCl	Corrosion occurred along side of wire that was facing O_2 inlet. Also corroded at bottom. Exposure time - 49 days.

TABLE D-1 (*continued*)

DESCRIPTION OF TEST	TEST ENVIRONMENT	RESULTS
Unstressed wire sample in vertical position. The top of the wire was below the solution-air interface and was surrounded by a plastic tube. O_2 was bubbled into the bottom of the cell. The bottom of the wire was encased in cement mortar where it entered the cell	Sat. $Ca(OH)_2$ + 3.5% NaCl	Corrosion started at the top of wire after a few hours. Extreme tuberculation growth developed and continued. Exposure time - 49 days
Same as above except N_2 was used instead of O_2.	Sat. $Ca(OH)_2$ + 3.5% NaCl with N_2	Slight corrosion started at top of wire within few hours but did not continue. Exposure time - 49 days.
Wires stressed to 225, 200, 175, 150 and 125 ksi.	3.5% NaCl Solution	Uniform corrosion on wires. Exposure time - 9 days. No failures.
Wires stressed to 225, 200 175 and 150 ksi.	3.5% $NaCl_2$	Uniform corrosion of wires. Exposure time - 36 days. No failures.
Wires stressed to 225, 200, 175 and 150 ksi. Stress-relieved wires (650°F - 1 hr). O_2 bubbled into bottom of cell.	Sat. $Ca(OH)_2$ + 3.5% NaCl with O_2	Failure at solution-air interface. Exposure time - 36 days. Two failures at 24 days
Wires stressed to 225, 200, 175, 150, 125 and 100 ksi.	Sat. $Ca(OH)_2$	No corrosion. Exposure time - 33 days
Wires stressed to 225, 200, 175 and 150 ksi. N_2 bubbled around top of wires. O_2 bubbled into bottom of cell	Sat. $Ca(OH)_2$ + 3.5% NaCl with O_2 and N_2	Corrosion at bottom of wires. No wire failures. Exposure time - 41 days.
Wires stressed to 225, 200, 175, 150, 125 and 100 ksi. N_2 bubbled into bottom of cell.	Sat. $Ca(OH)_2$ with N_2	No corrosion. Exposure time - 33 days. No failures.
Wires encased in 1/4 in. of cement mortar. Stressed to 200, 175,150, 125 and 100 ksi. O_2 bubbled into bottom of cell.	3.5% NaCl with O_2	Minor uniform corrosion. Exposure time - 49 days. No failures.
Wires stressed to 225, 200, 175, 150, 125 and 100 ksi. O_2 bubbled into bottom of cell.	Sat. $Ca(OH)_2$ + 3.5% NaCl with O_2	Corrosion evident on all samples. One failure at solution-air interface after 13 days. Exposure time - 19 days.

corrosive ions, such as chlorides, would also need to be present in the environment.

The potentiostatic measurements were clear in showing that even minor amounts of chloride in a simulated concrete environment are capable of destroying the passivation of the steel, thereby permitting it to corrode. Significant corrosion will occur under these conditions, however, only when differences in oxygen concentration are present at the steel surface.

Sodium nitrite was found to function as an inhibitor to steel corrosion in simulated concrete environments containing 3.5% NaCl but it was effective only when added in amounts greater than about 2%. Additional work is needed before the use of inhibitors, such as $NaNO_2$, could be recommended as an admixture for concrete. The cost factor alone might be a serious deterrent to its use. The current price of sodium nitrite is $0.24 per pound in 100-lb lots.

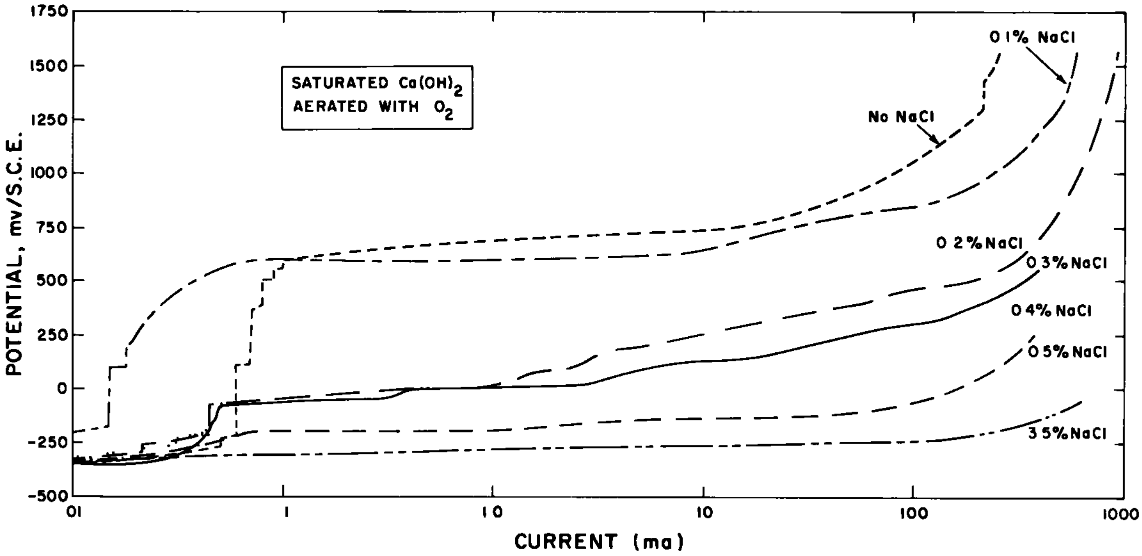

Figure D-3. Anodic polarization curves for prestressing steel in oxygenated $Ca(OH)_2$ solutions with varying amounts of NaCl.

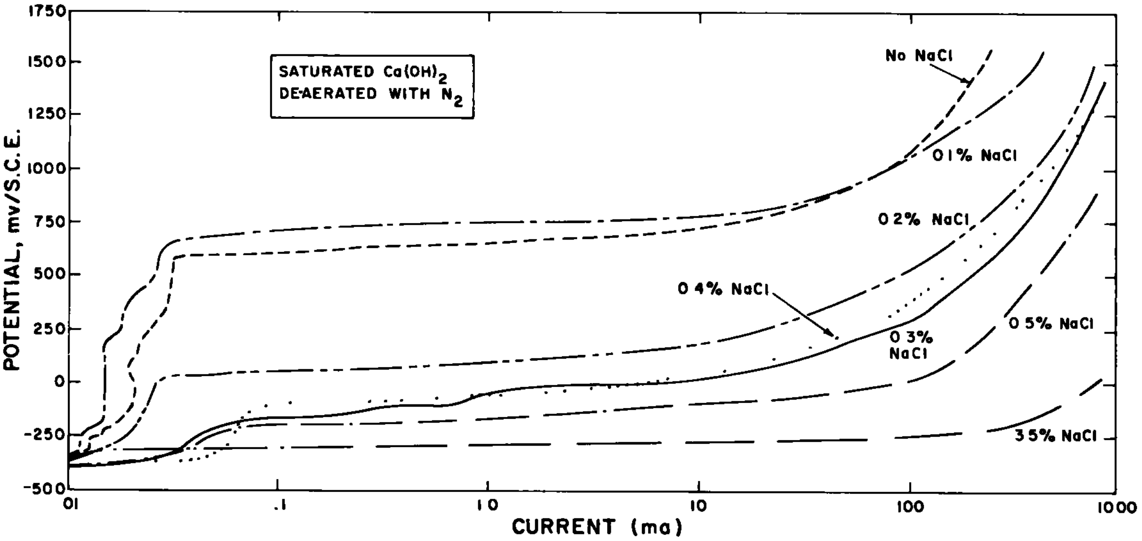

Figure D-4. Anodic polarization curves for prestressing steel in de-aerated saturated $Ca(OH)_2$ solutions with varying amounts of NaCl.

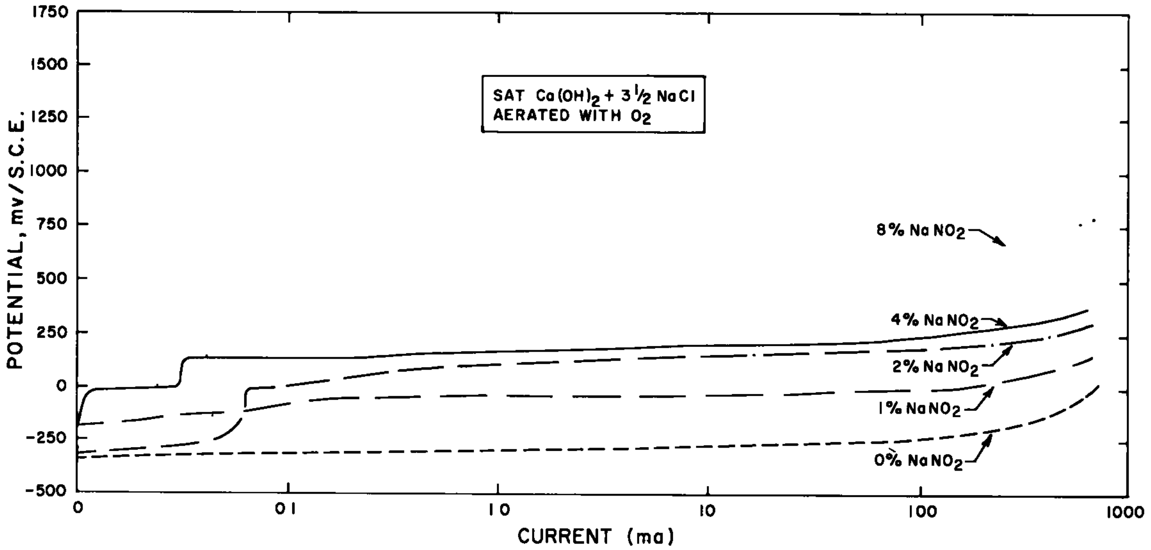

Figure D-5. Anodic polarization curves showing effect of various $NaNO_2$ additions on steel passivation.

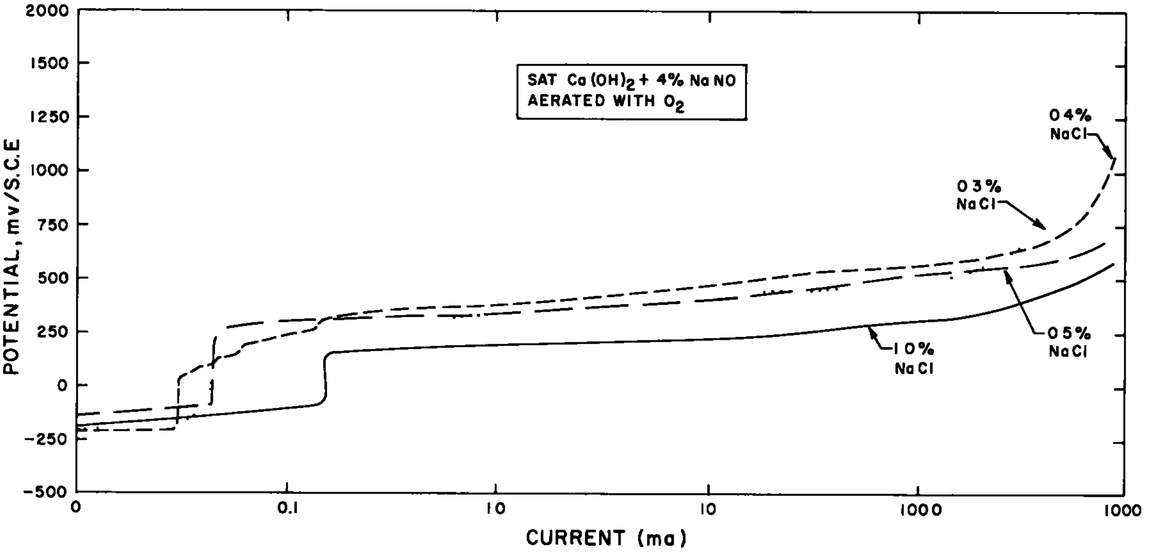

Figure D-6. Anodic polarization curves showing passivating effect of 4% $NaNO_2$ in saturated $Ca(OH)_2$ solutions containing varying amounts of NaCl.

APPENDIX E

PROTECTIVE COATINGS

PERMANENT-TYPE COATINGS FOR WIRE AND STRAND

If it was possible to devise a high-strength steel tendon that was immune to all types of corrosion in all possible environments, the concern over corrosion failures could be eliminated. Because a suitable coating applied to the tendon could conceivably accomplish this goal, one of the major efforts of the program was concerned with a study of possible protective coatings.

Analysis of the requirements of such a coating indicated that it should have the following properties:

1. No adverse effect on the strength or ductility of the steel.
2. Ability to withstand a wire elongation of 0.6% without cracking or spalling.
3. High bond strength to the concrete.
4. Ability to transmit shear from the steel to the concrete; also, no creep of a type that would partially relieve stresses in a pretensioned member.
5. Good long-term stability in a cement environment as well as in a cement environment contaminated by chlorides and other aggressive ions.
6. Sufficient wear resistance to withstand normal handling without loss of coating.
7. Sufficient flexibility to permit coated wires to be twisted into strand without spalling or cracking of the coating.
8. Sufficiently easy to apply so that good coverage and good bond to the steel could be assured.
9. No embrittling action on the steel from hydrogen pick-up during coating application.
10. Reasonable cost.

It became apparent as the program progressed that no single coating could meet all of these requirements. A porcelain enamel glass coating, for example, would be expected to provide the necessary durability, wear resistance, bond development to the concrete, shear transmittal, creep resistance, and, in addition, it could be applied at reasonable cost. Cracking and spalling would, however, be expected with a glass coating at steel elongations as high as 0.6%, and, perhaps even more important, the high firing temperatures required for bonding the glass to the steel (which are normally 1,000°F or higher) might cause a large loss in the strength of the cold-drawn steel. This loss in strength by heat treatment was confirmed early in the program when it was determined that a 5-min heating at 1,000°F reduced the ultimate tensile strength of 0.25-in. prestressing wire from 252,000 psi to 190,000 psi. Because such a reduction would appear to be unacceptable, no further work was carried forward on coatings of this type.

Silicate and phosphate-bonded coatings (which can be cured at much lower temperatures than porcelain enamels) would not suffer from this disadvantage. However, such coatings are brittle, and it is highly unlikely that they could withstand the required 0.6% elongation without excessive cracking and spalling.

This left only two other classes of coatings that appeared worthy of investigation; namely, the organics and the metallics. However, because of the large number of coatings that fall into these two categories, a further selection was necessary. According to the literature review, the two coatings that have shown the greatest promise for protecting prestressing steel are the hot-dipped galvanized and the epoxies. Therefore, these two coating types were given major emphasis in the testing and evaluation program.

The steel selected for coating evaluations was 0.25-in.-diameter post-tensioning wire of domestic manufacture having a typical chemical analysis of C 0.82, Mn 0.89, P 0.010, S 0.029, and Si 0.25. The wire, as received, had been cold-drawn and stress-relieved.

TEST PROCEDURES AND RESULTS

Strength and Elastic Properties

Electroplated coatings as well as the organics are applied at temperatures near ambient. No important changes in steel strength would be expected from the coating applications and none was found. Galvanized coatings, on the other hand, are applied by immersing the steel in a molten bath of zinc operating at a temperature of about 840°F. A treatment of this type would be expected to remove some of the temper from the steel, with a resulting loss of strength. To determine quantitatively the magnitude of such changes, an evaluation was made of the effect of various galvanizing treatments on strength, elasticity, and ductility.

For the strength tests, a group of 0.25-in.-diameter post-tensioning specimens, 24 in. long, were button-headed and divided into five groups of three each. One group was tested in the as-received condition and the other four after 0.5-, 1-, 2-, and 4-min immersion in a commercial galvanizing bath at 840°F. Figure E-1 shows the effect of time in the bath on both the ultimate tensile strength and the elastic limit. The elastic limit was arbitrarily taken as the stress required to cause a 40μin./in. plastic deformation in 10 min, as measured with an SR-4-type strain gauge.

The strength per unit area was based on the initial wire diameters rather than on the diameters after coating. This was considered a logical computation procedure because the only purpose for applying the coating was to provide corrosion protection. As shown in Figure E-2, the increase in diameter from the galvanizing treatments was fairly small. In the case of the 0.5-min immersion, for example, the wire diameter was increased by only 0.006 in. The thicknesses were measured on polished sections with a microscope.

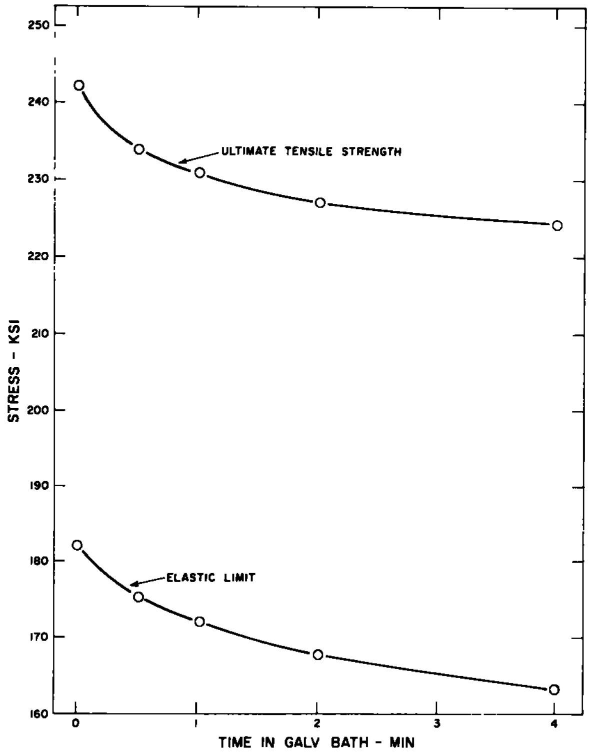

Figure E-1. Ultimate tensile strengths and elastic limits of 0.25-in.-diameter specimens. Elastic limit was arbitrarily taken as stress required to cause 40μ in./in. plastic deformation in 10 min.

Moduli of elasticity were determined using a mechanical gauge for the elongation measurements. The gauge length was 15 in.; the elongations were measured by means of a telescope with a micrometer eyepiece, with the load held constant at each stress level. Data obtained in this way showed that Young's modulus increased from 29.2×10^6 psi for the bare wire to 30.4×10^6 psi for specimens galvanized for 4 min. Computations for the galvanized specimens were again based on the wire diameter prior to coating.

Effect of Galvanizing on Steel Ductility

Figure E-3 shows the effect of the galvanizing treatments on wire ductility as measured by elongation after fracture. It will be noted that the longer treatments increase the ductility of the wire. No cracking of the galvanized layers was observed in any of these tests.

Effect of Galvanizing on Steel Relaxation

Prestressing steel relaxes or creeps when it is stressed at above about 50% of its ultimate tensile strength (*72*). The relaxation causes some loss in prestress, but this loss, together with the losses caused by concrete shrinkage, are normally compensated by increasing the initial prestressing load. If the wire or strand was galvanized to improve corrosion resistance, however, the steel relaxation might be appreciably higher and thus cause a much larger loss of prestress.

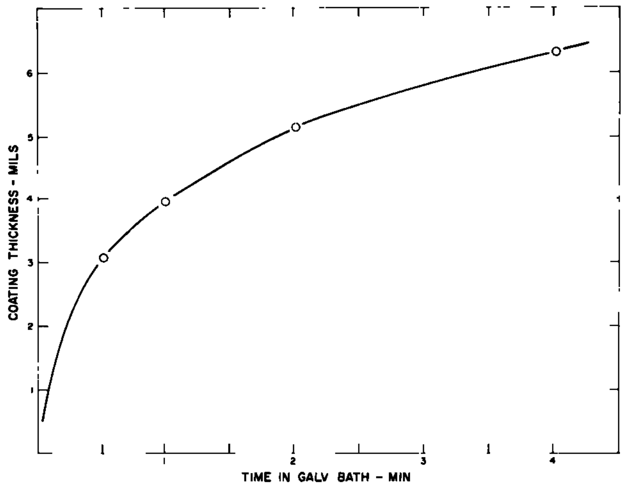

Figure E-2. Effect of time in galvanizing bath on coating thickness for 0.25-in. wires.

To determine if such is the case, measurements were made of the relaxation of both uncoated and galvanized 0.25-in.-diameter prestressing wire. The galvanizing was done for 0.5 min in a zinc bath at 840°F. The 0.5-min time was chosen as being typical of the immersion time used for galvanizing of wire in steel mills.

The relaxation was determined by first stressing a 12.75-in.-long specimen to the desired stress level in a Tinius-Olsen testing machine. The specimen was in a prestressing fixture during this stressing operation. When the selected stress level was reached, the load was transferred to the fixture by tightening two 1-in. nuts. The upper grip of the specimen rested on a calibrated load cell. Readings from this load cell gave both the initial load on the specimen in the fixture and the decrease in load with time.

Straight lines resulted when the percentage relaxation was plotted against log time over the time range of 1 to 100 hr. Typical data of this type are shown in Figure E-4. The tests were not continued beyond 100 hr. However, because in-

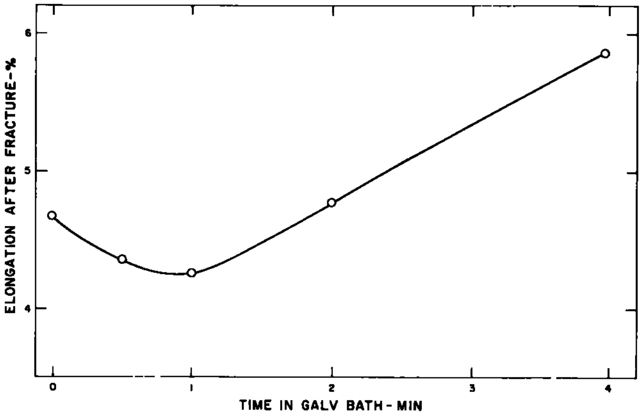

Figure E-3. Elongation after fracture for 0.25-in.-diameter galvanized prestressing wire.

terest is normally centered on relaxations after longer times, values for 1,000 hr were obtained by extrapolation of the curves, as indicated by the dashed lines in Figure E-4.

Figure E-5 is a summary of the test results. The relaxation of the galvanized wire is appreciably higher than that of the uncoated. At 70% prestress, for example, the relaxation for the galvanized wire is some 50% greater than for the uncoated. Or, stated differently, if an uncoated wire or strand was stressed at 70% of UTS, the galvanized could be stressed at only 60% for equal relaxations. This difference would, of course, not be as large if the relaxations for the galvanized wire were computed on the basis of the UTS of the galvanized rather than the uncoated steel. Such a computation does not appear justified, however, because in the intended applications the galvanized wire would normally be used in any given prestressed member as a direct substitute for uncoated wire.

Cracking and Spalling of Coatings During Simulated Prestress

A number of tests were conducted to determine the ability of coatings to withstand the steel elongations of approximately 0.6% that are encountered during prestressing.

The coatings tested were applied to the center 6 in. of 0.25-in.-diameter, 24-in.-long post-tensioning wires. The testing was done in a Tinius-Olsen testing machine using a head speed of 0.2 in. per min. Elongations were computed from the applied load using the previously determined modulus of elasticity of the steel. Cracking was detected in the metallic coatings by a pinging sound when cracking occurred; for the nonmetallics, visual examination of the coating surface with a magnifying lens was employed. The results of the testing are summarized in the following paragraphs.

Galvanized

No cracking of the galvanized layer occurred at an extension of 0.7% with any of the galvanized specimens, even for those with the heaviest layers (coated by immersion in the zinc bath for 4 min).

Electroplated Zinc

Several tests were made with specimens to which an electroplated zinc plating had been applied by a local jobbing plant. The electroplate was applied to acid-pickled specimens using a zinc cyanide bath at room temperature. The application thickness was 0.1 mil. In the testing, no cracks were detected after 0.7% extension.

Electroplated Cadmium

Cadmium was applied at a thickness of 0.6 mil, using standard commercial practices. The plated specimens all withstood 0.7% elongation without cracking.

Electroplated Nickel

A nickel coating was applied commercially at a thickness of 3.1 mil. The plating was done in a $NiSO_4$ solution at 140°F.

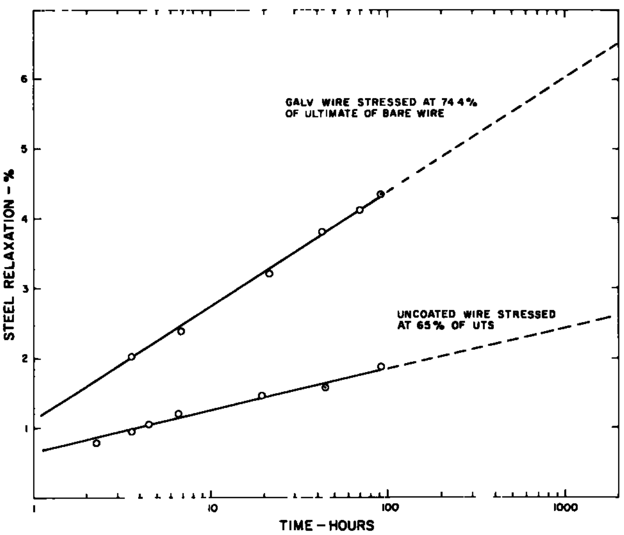

Figure E-4. Typical relaxation-time curves for 0.25-in. prestressing wire.

The resulting plate was brittle, and cracking of the layer was first detected at an extension of about 0.3%. In an attempt to overcome brittleness, attempts were made to apply a ductile nickel layer. Soft, ductile layers were achieved; however, in all of the trials, it was impossible to deposit a coating that was free of pin holes.

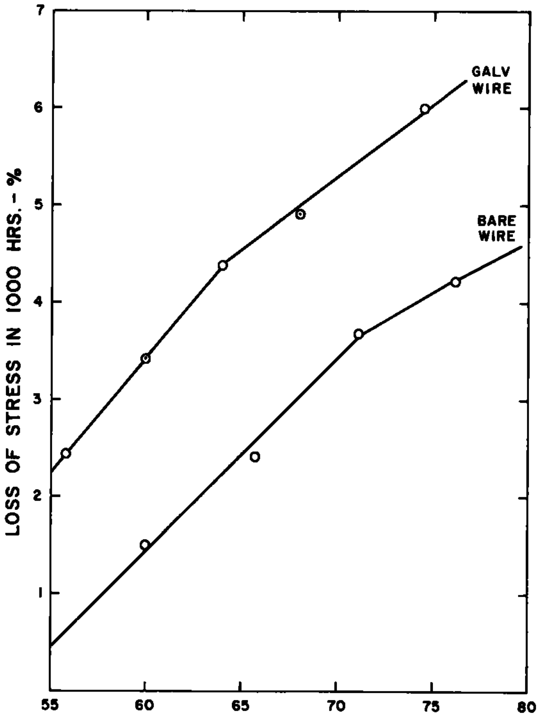

Figure E-5. Effect of galvanizing for 0.5 min. at 840°F on relaxation of 0.25-in. prestressing steel.

Epoxy Coatings

Several epoxy coatings were applied to the wire specimens by a local coatings application company. Difficulties were encountered in achieving a uniform application in all but one coating type. This was a light-brown epoxy, available from a commercial source, which was cured at 400°F for 2 hr. It was applied as two coats to a total thickness of 4 mils. No cracking occurred with this coating at a strain of 0.7%.

Summary of Cracking and Spalling Tests

The only coating of those tested that showed cracking in the simulated prestressing tests was the brittle electroplated nickel layer. It is possible that the electroplated zinc and cadmium might have behaved similarly if they had been applied at comparable thickness. Application of any type of electroplate to the high-carbon wires was found to be fraught with many difficulties by the local jobbing plant. In fact, these difficulties of obtaining good bond and continuous coverage were such as to indicate a relatively poor promise for the production application of electroplated coatings to prestressing steel.

The application problems with polymer-type coatings, although difficult, do not appear insurmountable. Galvanizing of the high-carbon wire was a simple operation and no problems were encountered in achieving satisfactory applications. Both the epoxy and the galvanized coating were found to be sufficiently flexible to withstand the elongations encountered during prestressing.

Corrosion Tests on Stressed Specimens

Tests with Coated Specimens in Aerated NaCl Solutions

A number of tests were made to determine the protectiveness of coatings while the steel is under stress. In the first tests of this type, 24-in.-long specimens of the 0.25-in. wire were placed in a specially designed fixture and stressed to the desired level in a tensile testing machine. After the stress level was reached, the load-application screws of the fixture were tightened to maintain the stress and the fixture was then removed from the tensile tester. An SR-4 strain gauge was attached to each specimen so that strain levels could be monitored throughout the testing.

A Lucite tube, 1.5 in. in diameter by 5 in. long, with rubber stoppers at each end, was placed around the specimen to contain the corrodent, which was a 3.5% NaCl solution. A tube inserted through the top rubber stopper was used to bubble air through the solution during the approximately 200 hr of testing.

Table E-1 summarizes the results obtained from this type of testing. With the exception of the electroplated nickel

TABLE E-1

CONDITION OF COATED SPECIMENS AFTER EXPOSURE TO AERATED 3.5% NaCl SOLUTION AT ROOM TEMPERATURE

SPECIMEN	COATING[a] THICKNESS (Mils)	STRAIN LEVEL[b] INITIAL (%)	FINAL (%)	TEST DURATION (Hrs)	DESCRIPTION OF SPECIMEN AFTER TEST
Uncoated	--	0.52	0 50	160	Heavy incrustation of rust, deeply pitted
Galv. - 1/2 min.	3.1	.46	40	192	White deposit on coating, no rust
Galv. - 1 min	4.0	51	.44	215	Same as 1/2-min. galv
Galv - 2 min.	5.2	51	46	215	Same as 1/2-min galv
Galv. - 4 min	6.3	44	38	192	Same as 1/2-min. galv
Galv. - 1/2 min (Part of Ctg. removed)	3.1	.46	41	214	Same as 1/2-min ; no rusting of exposed iron[c]
Electrolytic zinc	0.1	50	45	185	White surface deposit, no rust
Electro. cadmium	0 6	59	52	188	White surface deposit, no rust
Electro nickel	3 1	48	40	190	Rusting at circumferential cracks in coating
Epoxy Coating	4 0	47	45	214	No evidence of rust, no visible change in coating

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[a] Measured with a microscope on polished sections

[b] Determined with Type A-7 resistance-wire strain gauges.

[c] Exposed iron area was 3 inch long by 0. 08 inch wide Cathodic protection was active over entire area except where loose fragments of zinc corrosion product had deposited on exposed iron and apparently insulated the iron surface from the normal cell-current flow. Tiny rust spots were detected under these deposits

coating, which was cracked because of the high strain levels, all of the other coatings protected the steel from rusting under these highly accelerated test conditions.

Tests of Galvanized Specimens with Areas of Exposed Steel

One test was conducted to determine if the presence of exposed steel on a galvanized wire could result in an embrittlement failure because of cathodic charging of steel with hydrogen. Both Bird (*74*) and Tripler *et al.* (*95*) have stated that, when a break occurs in a zinc coating applied to steel, a galvanic couple is formed that generates hydrogen at the exposed steel cathode. Steel embrittlement could result if the hydrogen thus formed entered the steel.

Figure E-6 is a schematic of the test set-up that was used to determine if steel embrittlement would occur under these conditions. Prior to testing, a longitudinal strip, 3/32 in. wide by 3 in. long, of the galvanized coating was abraded away in the test region of both specimens that were tested so as to provide an area of exposed iron for possible cathodic charging.

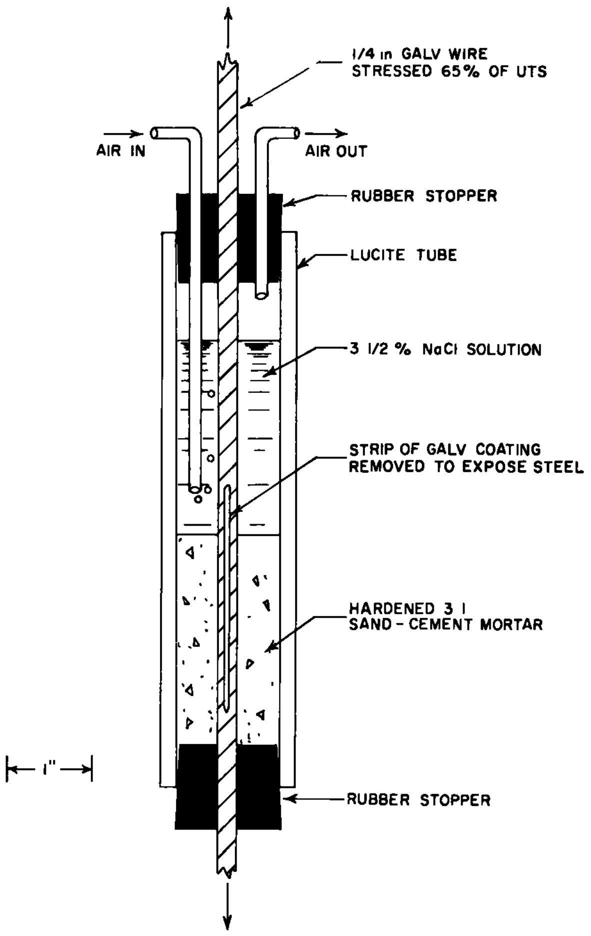

Figure E-6. Testing arrangement for evaluating hydrogen embrittlement damage for a galvanized wire with exposed iron.

Both specimens were stressed at 65% of UTS before vibration placement of a 3:1 sand-cement mortar. The load was then maintained at or near this level by a prestressing fixture. The mortar was moist-cured at room temperature for 28 days. At the end of this time, a 3.5% sodium chloride solution was added above the mortar in each specimen and air was then bubbled through the solution continuously at a rate of about 20 cc per minute.

After 6 months' exposure of the stressed wires to this environment, the two specimens were removed from the fixtures and examined. The results of this examination follow:

1. The zinc layer, which had an original thickness of 2 mils, had almost completely disappeared from the specimens in the region where they contacted the NaCl solution above the mortar surface. The specimen surface in this region was covered with a voluminous white deposit. The only rust staining of the deposit was near the mortar surface. However, no pitting of the steel was observed under the rust-stained deposit.
2. In the mortar-embedded region, both specimens showed slight rust formation at four or five anodic areas. Although the zinc layer had disappeared at these points, no pitting of the steel was observed.
3. The strip of exposed steel was protected cathodically because there was not evidence of corrosion of the exposed steel either where it was protected by the mortar or where it was in the NaCl solution above the mortar.
4. Tensile tests indicated no loss in strength or ductility from the 6-month corrosion treatment.

Because no hydrogen embrittlement failure occurred in these tests, and because there was no loss indicated in wire ductility after the corrosion treatment, it can be concluded that there was no appreciable cathodic charging of the exposed steel after the 6-month exposure to these very severe test conditions.

Corrosion Tests on Unstressed Specimens

Effect of Simulated Voids on Corrosion of Bare and Galvanized Specimens

Tests were conducted to compare the effect of simulated voids on the corrosion behavior of bare and galvanized specimens. A total of eight specimens were included in the test. Four were left uncoated and the other four were galvanized by immersion for 4 min in the zinc bath at 840°F.

Figure E-7 shows the testing arrangement. The mortar consisted of 3 parts by weight river sand to one part Type 1 portland cement. The water-cement ratio was 0.45. A small hand vibrator was used for compaction. The specimens were moist cured for 1 day and then oven-cured at 165°F for 16 hr.

Prior to casting, a single turn of ⅛-in.-diameter cotton twine was tied around the steel on four of the eight specimens so as to provide a simulated void. The twine was tied as a single turn 0.25 in. from the lower end of the rod.

The mortar cylinders, after curing, were partially immersed to a depth of 0.5 in. in a 3.5% NaCl solution for

1 day each week, air-dried for 2 days, and then stored in a moist atmosphere for 4 days. After 14 weeks of this cycling the mortar cylinders were split in half and the steel was inspected, with the following results:

Bare, No Void.—No rusting of the embedded wires was observed, except for the first ⅜ to 0.5 in. down from the top mortar surface. Near the top mortar surface, all natural-occurring voids were filled with corrosion products; at a depth of ⅜ in. from the top surface only a few of the voids were filled. Pitting of the steel occurred adjacent to the rust-filled voids. Maximum pit depth was 8 mils.

Bare, Simulated Void.—Rusting of wires occurred under the twine as well as at the top surface of the mortar. Three to four pits in the steel were observed under the twine in both specimens. Maximum pit depth was 3.5 mils.

Galvanized, No Void.—Little evidence of zinc corrosion was observed, even at the voids near the top surface of the mortar specimens, except for a slight whitening of the zinc.

Galvanized, Simulated Void.—The galvanized coating, which was 4 mils thick, was mostly converted to a white deposit under the cotton twine, but there was no evidence of steel corrosion in either specimen.

Summary.—The two conclusions that were drawn from this testing were:

1. The galvanized layer corrodes under the test conditions; hence, it would be expected to only delay the onset of steel corrosion and not prevent it.
2. Corrosion, when it occurred, was concentrated at voids between the steel and mortar.

Investigation of a Possible Gas-Forming Reaction Between Galvanized Coatings and a Cement Matrix

According to Bird (*101*) a gas-forming reaction can occur between galvanized steel and wet concrete. The reaction is reported to generate hydrogen, which causes the cement matrix adjacent to the steel to become spongy. The spongy layer has a low strength which, in turn, is reflected by low values in a bond test.

Although such a gas-forming reaction has never been reported in the U.S. when galvanized steel has been used in concrete, a few tests were believed desirable. Two types of tests were conducted. In the first type, one part by weight of Type 1 portland cement was dry-mixed with three parts of building sand. Mortars were prepared from the dry mixture using water/cement ratios of 0.45, 0.55, and 0.65. These mortars were then vibrated around wire specimens that had been galvanized for 0.5, 1, 2, and 4 min. After curing for 5 days at room temperature, the hardened mortar was grooved with a diamond saw and split away from the galvanized specimens. In all cases, a clean break occurred at the interface between the specimen and the mortar. Both a visual and a microscopic examination showed that there was no increase in the pore structure of the mortar adjacent to any of the galvanized steel specimens.

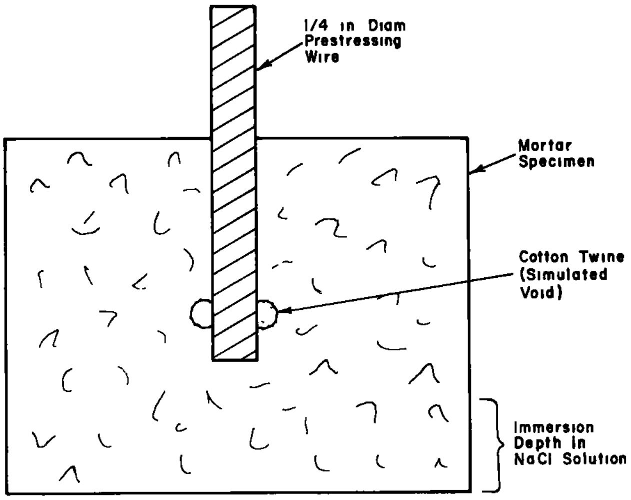

Figure E-7. Testing arrangement for evaluating corrosion protection of coatings on unstressed specimens.

In the second type of test, 50 grams of Type 1 cement were placed in each of three beakers; 50 grams of water were added to the first beaker, 100 grams to the second, and 150 grams to the third. Specimens galvanized for 0.25, 1, 2, and 4 min were then placed in each beaker. The interfaces between the supernatent cement extract and the specimen surfaces were examined periodically for a period of 3 days with a low-power microscope. No bubble formation was observed on the galvanized surfaces of any of the specimens. There was, however, a visible darkening of all of the galvanized coatings after a few hours contact with the cement extract.

Thus, the results of both tests indicate that no bubble-forming reaction occurs between galvanized steel and the Type 1 cement.

Bond Strength Measurements Between Coated Steel and Concrete

One of the most important requirements of a protective coating for prestressing steel is that it shall not detract seriously from the bond that normally develops between the steel and concrete. In pretensioning, especially, it is important that the steel be well bonded; otherwise, there could be a serious loss of prestress. In post-tensioning, good bonding to the grout is important not only for structural stability but also to provide bonding of tendons in case of a failure at the grip.

Two types of bond test were used during the course of the investigation. In the first type, a determination was made of the compressive force required to break the bond between 0.25-in. wire specimens and a hardened sand-cement mortar mix. Figure E-8 shows the fixture that was used. The 0.25-in.-diameter steel specimen, which was 3.5 in. long, was held in a true vertical position during casting by a brass mandrel at the bottom. The specimen passed through an oversized hole in the mandrel and a small amount of caulking material was placed in a chamber at the top of the hole to prevent leakage. During casting, the specimen was held tightly against the hole wall by two setscrews placed 90° apart. After curing, but prior to test,

Figure E-8. Bond-test fixture used in early testing.

these setscrews were loosened. The fixture was then placed in a testing machine where the compressive load required to shear the specimen from the hardened mortar was determined. All tests were made with a testing machine head movement of 0.0133 in. per min.

The mortar consisted of one part by weight of Type 1 portland cement and three parts of building sand. The w/c ratio was 0.45. A small hand-held vibrator was used during placing of the mortar. The specimens were moist-cured at room temperature for 24 hr; next, oven-cured at 160°F for 15.5 hr; and then aged for an additional 3 days at room temperature.

Table E-2 shows that the average bond strength for the galvanized specimens as measured by this procedure was about 45% higher than the average for the bare specimens (1,058 against 723 psi). The scatter, however, for the galvanized specimens was almost twice as great.

The reason for the greater bond strength for the galvanized specimens can probably be ascribed to (1) the somewhat greater roughness of the galvanized surfaces, and (2) the slight chemical reaction that occurs between a galvanized coating and wet cement. When the mortar specimens were split open after test, so as to expose the steel, it was found that the galvanized layers were discolored wherever contact had occurred with the mortar. Also, the galvanized specimens had somewhat more mortar adhering to their surfaces than did the bare specimens. Both specimen types, however, gave relatively clean fractures between the specimen surfaces and the mortar.

TABLE E-2

BOND STRENGTHS OF BARE AND GALVANIZED SPECIMENS AS DETERMINED WITH APPARATUS SHOWN IN FIGURE E-7

DESCRIPTION [a]	SPEC.	LOAD TO CAUSE FAILURE OF BOND (LB)	COMPUTED SHEAR STRESS (PSI)
Bare, as received	B-1	1080	680
	B-2	1004	633
	B-3	1194	752
	B-4	1300	820
	B-5	1156	728
Avg.		1147	723
Std. dev.		98	62
Coef. of Var.[b]		8.6%	8.6%
Galvanized	G-1	2060	1249
	G-2	1870	1132
	G-3	2130	1290
	G-4	1500	910
	G-5	1320	800
	G-6	1900	1151
	G-7	1420	860
Avg.		1747	1058
Std. dev.		296	179
Coef. of Var.[b]		17.0%	17.0%

[a] Length of specimen embedded in mortar was 2.0 in. Specimen diameters were 0.250 in. for the bare, and 0.262 in. for the galvanized.

[b] Coefficient of variation (standard deviation expressed as a percentage of average).

It was believed that the data obtained from the compressive-type test were of questionable value because the test did not simulate the stress conditions that exist in prestressed concrete. Therefore, a second testing procedure was devised. Figure E-9 shows the test fixture that was used. The specimen (which was 0.25 in. in diameter by 12.5 in. long) was first stressed to 7,000 lb (150,000 psi) in a testing machine, and the 1-in. nuts on the two threaded rods were tightened to transfer the load from the machine to the fixture. The fixture was then removed from the testing machine and a block of mortar (3:1 sand-cement, w/c ratio 0.40) was cast around the center section in a Lucite mold using a hand-held vibrator for compaction. The size of this mortar block was 2.5 × 2.5 × 5 in. The length of the block was purposely kept small so that bond slippage would occur in the later testing.

After an oven cure of 16 hr at 160°F the mold was removed, an SR-4 wire strain gauge was epoxied to one surface (Figure E-9), and the specimen, while still under stress in the fixture, was dried at 150°F for 24 hr so as to minimize shrinkage effects during testing.

After this drying treatment, the fixture was placed back in the testing machine and the specimen was loaded very slowly to a level where the 1-in. nuts could just be released. Release of these nuts transferred the tensile loading of the wire from the fixture to the testing machine. The machine-indicated load on the wire was next decreased in 200-lb increments and the corresponding strain on the mortar surface was recorded.

Figure E-10 shows the type of curves that were obtained when coated and uncoated 0.25-in. wire specimens are tested in this manner. The load corresponding to Point "A" on each curve was designated as the bond slippage load. Note that for wires grit-blasted with —80 mesh glass beads at an air pressure of 80 psi no bond slippage load was observed, indicating there was no bond slippage occurring, even up to the maximum loading of 6,800 lb.

Similar tests were made with 0.5-in. strand. The size of the mortar block was the same as for the 0.25-in. wire. Also, the same mortar, curing cycles, and testing procedures were used.

Results with the strand are plotted in Figure E-11. Consider first the middle curve for uncoated strand with smooth wires. The curve shape suggests that slippage was beginning to occur between the hardened cement mortar and the outer surfaces of the wires at a load of about 2,300 lb (Point A). This slippage continued to occur until Point B was reached. At this load, the spiral serrations apparently picked up the bonding. However, this mechanical interlock type of bond did not appear as effective as the earlier bonding, because the curve beyond Point A has a lower slope.

As indicated by the upper curve, grit blasting of the outer surfaces of the wires with glass beads greatly enhances the bond between the wire surfaces and the mortar. Initial bond slippage in this case did not occur until the load level was well above 7,000 lb. On the other hand, coating the strand with a corrosion-inhibiting oil (a description of which is included in a later section) permits initial slippage to occur at a load of only 300 lb. Note also that the slope after

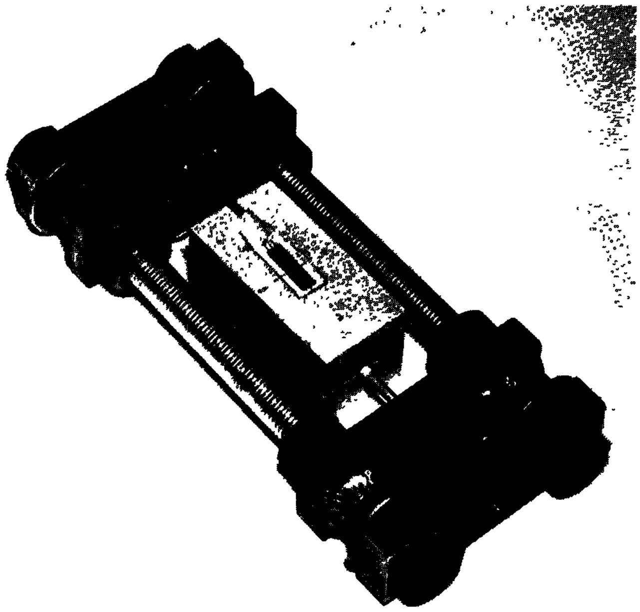

Figure E-9. Fixture used for determining bond release loads.

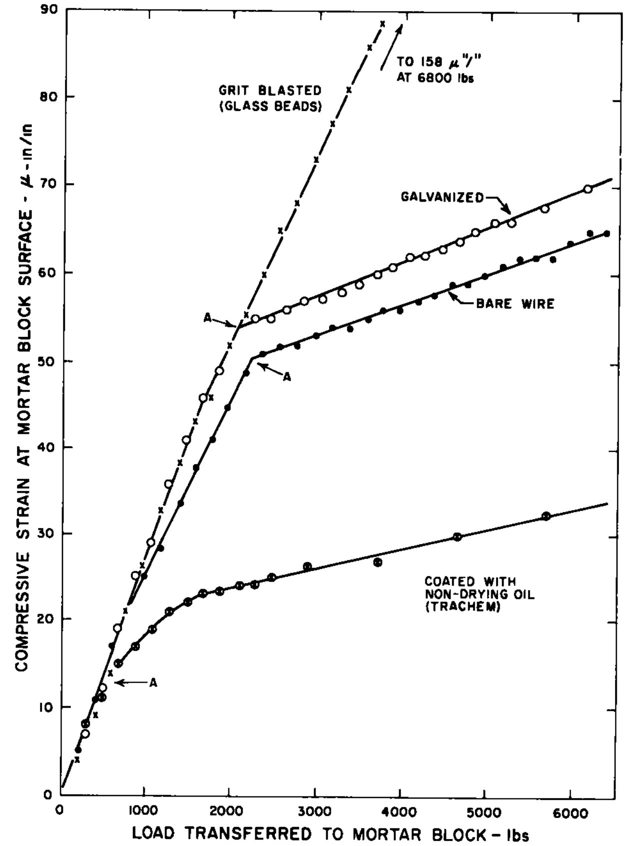

Figure E-10. Bonding behavior of 0.25-in.-diameter prestressing steel. Inflection points, designated "A," were taken as the bond release loads.

Point B for the oil-coated strand was appreciably lower than for the uncoated.

The reproducibility of the test was reasonably good. With bare wires, for example, bond slippage loads for three specimens were 2,200, 2,500, and 2,300 lb. The bond slippage loads for galvanized specimens appeared to be dependent on the duration of room-temperature cure. One galvanized specimen cured for 4 days at room temperature before oven curing, so as to permit chemical attack between the galvanized layer and the cement matrix, showed a bond slippage load of 3,600 lb. Two similar specimens that were allowed to cure at room temperature for only a few hours before oven cure (which was the standard test procedure) gave values of 2,200 and 2,100 lb.

Table E-3 is a summary of the data from the bond slippage tests. Of the possible permanent-type coatings listed, only the galvanized and the epoxy developed bonding that was comparable to the bare steel. The tar-epoxy coating * is reasonable in cost and has been reported to provide excellent protection for steel, especially when applied as two coats at a total thickness of 0.015 to 0.020 in. That this coating would be unsuitable for prestressing steel because of bonding deficiency is indicated by the extremely low bond release loads for the coated specimens (Table E-3). The value for the first specimen was only 50 lb, whereas for the second it was zero. Examination of the interface between the coating and the hardened mortar after testing showed that the mortar in contact with the coating was discolored and somewhat soft. This could explain the lack of bond development. Another possibility is that the coating itself, although seemingly hard, is subject to flow and is therefore incapable of transmitting the shear stress to the steel.

Bond release load data for temporary-type coatings are also included in Table E-3. These are discussed in a later section.

Wear Resistance of Coatings

Any protective coating applied to prestressing wire or strand would be subjected to wear during normal handling operations. The dragging of steel along prestressing beds is not an uncommon practice and in post-tensioning operations it would be difficult, if not impossible, to place the tendons in ducts without a great deal of scraping of the steel surface against the duct lining. Wear in this case, however, would not be as severe as when steel was dragged along concrete or hard soil surfaces, and the possibility of this sort of treatment occurring during prestressing operations is always present.

Because of the need for a coating to withstand this type of wear, a test was devised in which coated wires were subjected to an abrading action in contact with concrete. The equipment design, which was relatively simple, is shown schematically in Figure E-12.

A small cylindrical mortar specimen, 2.75 in. in diameter by 2 in. long, was used as the abrading medium. This cylinder was prepared with three parts by weight sand to one part by weight Type 1 portland cement. The water-

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* Tarset, marketed by the Chemical Division, U.S. Steel.

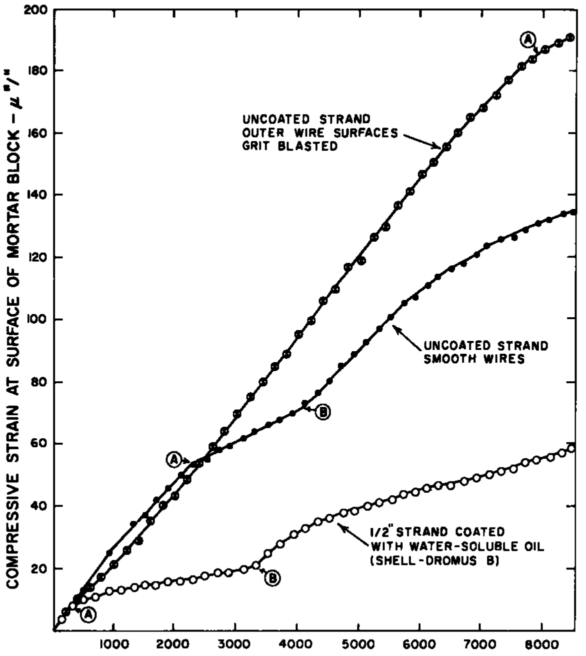

Figure E-11. Bonding behavior of 0.5-in.-diameter strand. Inflection points, marked "A," denote loads at which bond slippage occurs on outer wire surfaces; points marked "B" indicate where pick-up of bond begins to occur by mechanical gripping between spiral serrations of strand and the hardened mortar.

TABLE E-3

SUMMARY OF BOND SLIPPAGE TESTS

SPECIMEN	COATING THICKNESS (MILS)	BOND SLIPPAGE LOAD[a] (LB)
Uncoated, smooth	0.0	2330
Uncoated, grit-blasted	0.0	7600
Galvanized	4.0	2050[b]
Epoxy, painted[c]	3.0	1500
Epoxy, sprayed (one ct.)[c]	2.5	2100
Epoxy, sprayed (two cts.)[c]	7.6	2100
Epoxy, sprayed (grit-blasted surface)[c]	8.0	2100
Tar-epoxy ctg.[d]	10.0	25
Sod. silicate ctg.[e]	0.1	2150
Shell water-sol. oil ctg.	<0.1	300
Trachor, dry-film ctg.	2.0	1100
Trachor, non-drying ctg.	3.0	900
Shell VPI[f]	—	2200

[a] Each value is average of two or more specimens.

[b] Mortar block aged overnight at room temperature before oven cure. Significantly higher values result when room-temperature cure is extended for several days.

[c] Epoxy coating No. 9335, Rust-Oleum Corp., Evanston, Ill.

[d] Tarset coating, U.S. Steel Co., Pittsburgh, Pa.

[e] DRI coating S-65.

[f] Specimens sealed in tube with Shell VPI No. 250 crystals for 4 days prior to testing.

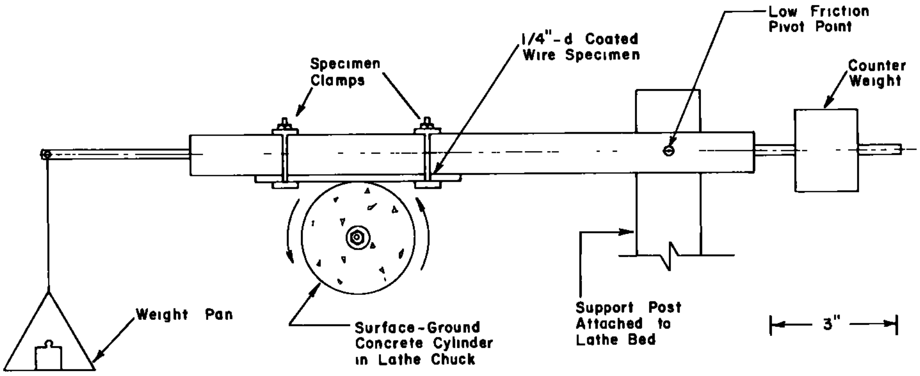

Figure E-12. Equipment for abrasive-wear test.

cement ratio was 0.45. The cylinder was cured at 165°F for 16 hr, after which it was aged for 75 days at room temperature. Casting was done in a brass mold using a small hand vibrator for compacting.

Prior to use, the cylinder was placed in the lathe and trued with a carbide tool. The surface after this treatment contained small open pores, and, except for the absence of aggregate, it resembled closely the appearance of a concrete surface that had been subjected to fairly severe wear conditions. The cylinder was resurfaced after each test.

Figure E-13 shows typical results obtained for coated and uncoated 0.25-in.-diameter prestressing wire specimens that were subjected for 1 min to various load levels. In these tests, the cylindrical specimen was turning at 42 rpm, which corresponds to a surface speed of 30 ft/min.

The data plotted in Figure E-13 show that uncoated prestressing steel has a considerably higher resistance to the abrasive wear of the type described than does the galvanized coating. Both, however, are many times more resistant to the test conditions than the commercial brown epoxy coating that was tested. Other epoxy coatings behaved similarly to the brown type. Incorporating glass fabric into an epoxy coating reduced wear rate by a factor of two, but the rate was still many times higher than for galvanized. One epoxy cement used for cementing strain gauges was troweled on a wire specimen and the layer was then cured at 400°F for 2 hr. The wear rate of the resulting layer was comparable to galvanized; however, because of the highly viscous nature of the cement, it would be extremely difficult, if not impossible, to apply such a composition as a uniform coating layer on wire.

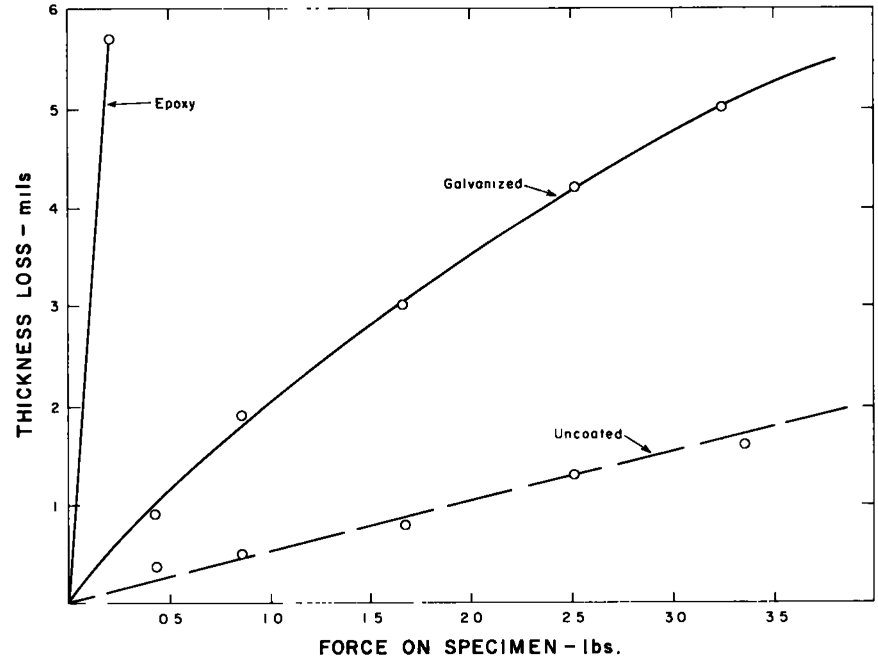

Figure E-13. Typical wear-test results for 1-min. abrasion times.

SEALANT-TYPE COATINGS FOR CONCRETE SURFACES

Because the conclusion from the literature review (Appendix B) was that the application of impermeable coatings to prestressed concrete bridge girders was not a promising approach for protecting steel against corrosion, no experimental work was conducted on coatings of this type.

It is believed obvious that over-all construction costs would be appreciably increased if all girders on a prestressed concrete bridge were required to have one or more applications of a relatively expensive coating, such as an epoxy. Also, such a requirement would detract from one of the major advantages of prestressed concrete construction (i.e., that it is largely maintenance-free and does not require periodic painting). A sealant-type coating on a concrete girder would protect the steel against corrosion only if the coating was impermeable to air and moisture, well-bonded to the concrete, and free of coating flaws. In addition, it should retain these properties throughout the service life of the structure. It is rather difficult to conceive of such long-time integrity with any of the coating systems now available.

COATINGS FOR PROTECTION OF PRESTRESSING STEEL DURING SHIPPING AND STORING

Because prestressing steel is always used at high stress levels, any appreciable corrosion that occurred before steel embedment could be dangerous, because the corrosion could cause a decrease in the load-bearing capabilities of the tendon.

Light superficial rusting is probably not detrimental to steel performance, and, in fact, its presence might even enhance bonding. Nevertheless, the mere presence of rust automatically informs the inspector that the steel has been exposed to corrosive conditions. Because it is not practical to inspect the entire lot of wire or strand for pitting due to possible local cell action, his natural inclination is to reject the entire lot. Also, in grouting operations, serious corrosion of steel can occur if the tendons are placed in the ducts weeks or even months in advance of the grouting. Under such conditions, protection against corrosive damage is mandatory.

Thus, for the reasons cited, one part of the coatings research was devoted to protective systems that would assure rust-free steel at the time of its encasement in either concrete or grout.

Experimental Procedures

Two test procedures were employed to evaluate the effectiveness of temporary-type coatings. The first procedure consisted of:

1. Applying experimental coatings to the lower half of 2.5-in.-long specimens of 0.25-in.-diameter prestressing wire.
2. Hanging the specimens on a rack.
3. Placing the rack with specimens attached in a desiccator containing water in the bottom.
4. Periodically placing the specimens and rack in a refrigerator at 40°F for 1 hr before reinserting them into the desiccator.

The insertion of the cooled specimens into the desiccator caused moisture to condense on the specimen surface. The surfaces then remained moist for extended periods. The test procedure adopted was to subject the specimens to the cooling cycle once each 48 hr.

In addition, to test the effectiveness of the coatings in preventing rusting in marine environments, a modification of the test procedure described was used in which specimens were subjected to a single immersion in 3.5% NaCl solution once during each coating cycle. Similarly, to simulate the effect of the rinsing action of rain, some specimens were immersed in distilled water during each cooling cycle.

The second evaluation test for temporary-type coatings was a measurement of the bond slippage load through use of the same test procedures described in the preceding section on the permanent-type coatings. In including this test it was assumed that a temporary coating that interfered with bond development to the steel would be unacceptable, no matter how well it protected against steel corrosion. It was also assumed that elaborate coating removal methods would be unacceptable. Hence, only a water rinse was employed before embedding the specimens in the mortar blocks that were needed for the determination of bond release loads.

Description of Coatings and Test Results

Sodium Silicate Coating

Exploratory work showed that thin films of sodium silicate provided rust protection for steel in moist environments. In this work, films were formed by dipping specimens into aqueous solutions containing 4, 10, 20, 40, 50, 75, and 100% by volume of "N" brand sodium silicate,* followed by drying in air. The "N" brand is a water solution containing 37.6% by weight of a sodium silicate with a Na_2O-to-SiO_2 weight ratio of 3.22.

Experiments were also made in which known rust inhibitors (such as chromates and nitrites) were incorporated as additives to the silicate solutions. At the conclusion of the exploratory work, a single composition was selected on the basis of comparative performances in moisture condensation tests. This composition, which was designated S-65, consisted of the following:

COMPOSITION OF COATING S-65	PARTS BY WEIGHT
"N" brand sodium silicate solution	70
Water	50
Sodium nitrite	5

Specimens were coated by immersion in the S-65 solution, followed by drying in air. Film thickness after drying was approximately 0.2 mil.

A summary of the corrosion-test results with the S-65 coating is included in Table E-4. Figure E-14 shows the appearance of the coating after exposure to the moisture condensation conditions. The lighter areas on the coated part of the specimen are probably caused by precipitation

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* Available from Philadelphia Quartz Co., Philadelphia, Pa.

TABLE E-4

RUSTING TIMES FOR WIRE SPECIMENS WITH TEMPORARY-TYPE COATINGS

COATING DESCRIPTION	COATING THICKNESS (MILS)	TIME FOR FIRST RUSTING (HR), BY TEST CONDITION		
		A[a]	B[b]	C[c]
Sod. silicate ctg. S-65	0.2	>1100	360	75
Sil. ctg. S-65, two cts.	0.3	>1100	440	120
Sil. ctg. S-65, two cts., with abraded strip	0.3	>1100	380	75
Shell Dromus B ctg.	<0.1	>1100	720	240
Dromus B ctg., with abraded strip[d]	<0.1	24	< 24	< 24
Trachem dry-film ctg.	3.5	>1100	>1100	640
Trachor non-drying ctg.	4.0	>1100	>1100	450

[a] Test conducted in high-humidity environment with twice-weekly condensation of moisture on coating surface by specimen cooling.

[b] Same treatment as [a], except immersed in water prior to each specimen cooling cycle.

[c] Same treatment as [a], except immersed in 3.5% NaCl prior to each specimen cooling cycle.

[d] Film aged 95 days before abrading treatment.

of SiO_2 from the sodium silicate film by the action of CO_2 in the atmosphere. No adverse effect on coating protection was noted from this reaction.

That S-65 film still protects after a loss of coating by mild abrasion is indicated by results of one of the tests included in Table E-4. It is probable that the protective action of the silicate film is due to its slow solution in condensed moisture, thus creating high pH conditions and thereby passivating the steel surface. If the moisture film was sufficiently thin and continuous the entire film would achieve this high pH and, as a result, the steel would be protected, even though breaks occurred in the coating layer.

The effects of the presence of the S-65 coating on bond development to a cement matrix are given in Table E-3. It will be noted that the coating had only a minor deteriorating influence on bond behavior.

Water-Soluble Oil Film

As indicated in Table E-4 and Figure E-13, fairly good protection to rusting in the moisture condensation test was obtained by applying a "water-soluble" oil film to the steel specimens.* Its main application is as a cutting oil. It is not truly water soluble; rather, it forms an emulsion with water. The film that forms when the oil is applied to steel wires by dipping is very thin (less than 0.1 mil). The film, when first applied, can be removed with water; however, after a few hours of aging it loses this property and it must be removed with a solvent. The protectiveness of the oil film was found to be superior to the silicate coating for the specimens immersed twice weekly in either water or NaCl solution.

The Dromus B oil film, on the other hand, showed three undesirable characteristics: (1) on aging it lost its rehealing properties, permitting rusting to occur once the film had been damaged by abrasive wear (Table E-4); (2) it was not removable by a water rinse after aging; and (3) its presence on the steel prevented satisfactory bond formation (Table E-3).

Organic-Type Coatings

Two coatings marketed by Tracor, Inc., Austin, Texas, were tested. One was a product called Trachem Lubecoat. This is a water-displacing, corrosion-inhibitor type coating that imparts lubricating properties to the metal surface. The coating (which can be applied by brushing, dipping, or spraying) remains soft and tacky after application.

The second coating was a product called Trachem Drycoat. This dries rapidly to a clear, non-tacky film. It is intended as a corrosion-inhibiting film on metal parts. According to the manufacturer, it can be easily removed with any organic solvent. Recommended application thickness is 4 mils.

Both coatings were applied to specimens and subjected to the moisture condensation test. As indicated in Table E-4, both provided good protection against rusting. Table E-3, however, shows that both coatings have an adverse effect on bonding.

Shell Vapor-Phase Inhibitor

A solid amine nitrite in the form of fine crystals is currently in limited use for protecting prestressing steel during shipping and storing. This material (marketed by the Shell Oil Co.) sublimes slowly during use, producing vapors which act in the presence of moisture and oxygen to prevent rusting of the steel. To function effectively, the steel should be in a sealed enclosure. During shipping, this is accomplished by placing a small amount of crystals next to the steel and then wrapping the shipment so that it will be protected against air circulation.

The VPI crystals have also been used for protecting steel in post-tensioning ducts prior to grouting. For this application, the crystals are sprinkled on the tendons as they are

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* This material is marketed by the Products Application Division, Shell Oil Co., Chicago, Ill., under the name of Dromus Oil B.

Figure E-14. Specimens after 800 hr in moisture condensation test. Lower half of Specimen 88 is coated with Shell water-soluble oil; lower half of Specimen 92, with sodium silicate-sodium nitrite coating, S-65.

pulled through the ductwork. The ends of the ducts are then closed off to prevent excessive amounts of moisture from entering. The crystals are removed by flushing the duct with water prior to the grouting operation.

Because of the way in which the VPI crystals are used, the moisture condensation test mentioned earlier was not an acceptable testing procedure. Instead, experiments were performed in which 24-in.-long, 0.25-in.-diameter prestressing wires were placed in 1-in. I.D. Pyrex tubes. The tubes were inclined at 5° while resting on a flat surface. Before sealing the tube ends, 20 ml of water were placed at the lower end of the tube and 5 grams of Shell VPI No. 250 crystals were placed at the top end.

Examination of the wires after 50 days' exposure to this environment showed that rust spots on the wires extended from the lower end to within 1 in. of the VPI crystals. No tests were made with chloride solutions, because there would be little likelihood of chlorides entering any of the shipping packages or ducts into which the VPI crystals are placed.

Data included in Table E-3 show that the bonding of steel to concrete is not affected by prior contact for 6 weeks with the VPI crystals.

SUMMARY AND RECOMMENDATIONS ON PROTECTIVE COATINGS

Stringent requirements apply to prestressing steel coatings, and the perfect coating does not exist. Nevertheless, accelerated tests have shown that galvanized coatings provide some of the corrosion protection that might be needed when prestressed girders are used in chloride environments. The zinc layer, although not completely inert in environments of this type, has been shown to appreciably extend the time before pitting of steel occurs.

No hydrogen embrittlement occurred during corrosion testing when exposed areas of iron were present on galvanized specimens. Somewhat lower prestressing loads would need to be used with galvanized tendons. However, in severely corrosive environments this would probably be a small price to pay for the increased corrosion resistance.

One practical advantage to the use of galvanizing for increased corrosion protection would be that the steel mills producing prestressing steel already have facilities and the engineering know-how for the hot-dip galvanizing of wire and strand. It is estimated that the galvanizing would add about 10 to 15% to the cost of the steel.

Epoxy coatings were found to provide better corrosion resistance for prestressing steel in chloride-containing environments than did galvanizing. Only short-time tests were conducted. However, the absence of corrosive attack under the highly accelerated conditions suggests that the epoxy coatings are capable of providing good protection for the steel over a period of many years. Unfortunately, epoxy coatings suffer from the following deficiencies when used on prestressing steel:

1. They are barrier-type coatings and, therefore, do not provide protection once the coating is damaged.
2. They are difficult to apply as a uniform and continuous layer.
3. They lack wear resistance to the type of handling operation encountered in prestressing plants.
4. They are expensive.
5. A major engineering research and development program would probably be required before satisfactory coating applications could be accomplished in a steel mill.

In view of these deficiencies, it is recommended that, at present, epoxy coatings not be considered for protection of prestressing steel. The same recommendation also applies to any of the currently available organic-type coatings. Also, because electroplated coatings are difficult to apply with good continuity and good bonding to the high-carbon prestressing steel, and because of the probable high cost of applying them, their use for protecting the steel against corrosion is not recommended.

In addition, protection of prestressing steel by application of a sealant-type coating to the concrete surfaces of bridge girders is not a promising approach. A coating of this type should be low in cost and easy to apply, should develop good bond to the concrete, should be impermeable to moisture, should have excellent durability to weathering, and should be reasonably free of maintenance. Unfortunately, coatings that meet all of these requirements are not currently available.

Two protection systems are recommended for minimizing rusting of prestressing steel during shipping and storing:

1. The use of crystals of a vapor-phase inhibitor placed in the shipping container and also in sealed-off ducts in those cases where post-tensioning tendons need to be placed in the ductwork weeks or months in advance of grouting.
2. The application of thin films of a sodium silicate-sodium nitrite coating at the steel mill. The cost of either system is nominal. The VPI crystals are reasonable in cost, and only small amounts are needed. The cost of the "N" brand sodium silicate is only about $0.50 per gallon in bulk lots. With the sodium nitrite added, the cost per gallon would be about $0.70.

Finally, the researchers believe that coating of prestressing steel is an unnecessary precaution for the great majority of prestressed bridges, because adequate protection can be provided by good-quality concrete with adequate cover. The absence of steel corrosion in the pretensioned pile fragment exposed for 11 years to twice-daily immersion in sea water (Appendix H) is good evidence that prestressed concrete is capable of providing the necessary protection for the steel, even in a chloride-containing environment. In fact, the only bridge structures in which coating of the prestressing steel might be called for would be those high-cost structures over sea water where some added assurance of an extra long service life would be desirable. Galvanized coatings, although not completely inert in a chloride-containing concrete environment, would help to provide this assurance.

APPENDIX F

CORROSION TESTING OF PRETENSIONED BEAMS

Because it is sometimes difficult to analyze the results of corrosion experiments made in the laboratory and to predict with full assurance what will occur in an actual girder, some corrosion testing was carried forward using commercially fabricated pretensioned beams.

The test was designed to provide information on the following variables:

1. Effect of voids between concrete and steel.
2. Effect of concrete cover.
3. Effect of live loads.
4. Effect of sizeable tensile cracks present in the concrete.
5. Effect of accidental overloading (i.e., cracking of the beam followed by closing of the cracks by load reduction).

The selected corrosion environment consisted of a 3.5% by weight solution of NaCl. This solution was held in corrodent cells which were sealed against the beam surface at the area of maximum applied tension. Close control of test conditions from beam to beam was believed imperative for meaningful comparisons; hence, a decision was made to use controlled exposure conditions rather than to expose the beams in natural corrosive environments, such as by partial immersion in sea water at various coastal locations.

The selected method of subjecting the beams to sodium chloride solution only at the midspan simulates the type of exposure that might be encountered frequently in bridge girders. Deicing salts permeating from the deck through to the girder would in most cases reach the girder at only a few points. Likewise, on bridges over salt or brackish water more chlorides would be expected to reach the girders near the end abutments where air-borne sprays from the wave action would reach their highest concentration.

The time limitations of the contract prevented long-time exposure. However, even in exposure times as short as 1 year, the beginnings of steel corrosion can often be observed, and this information can then be used to predict with fairly good reliability what might happen after prolonged exposures.

BEAM FABRICATION

The beams were fabricated in the summer of 1967 at the Denver plant of Prestressed Concrete of Colorado, Inc. All 16 beams were cast in line in a single bed in treated plywood forms. Bulkheads at the end of each form were used to position the two ⅜-in.-diameter prestressing strands. The ⅜-in. strand was selected rather than the more commonly used 0.5-in. diameter so as (1) to provide a member with lighter weight, and (2) to accelerate the effects of any corrosive attack on beam behavior.

The beams were cast using the following concrete mix:

INGREDIENT	PARTS BY WEIGHT
Type 1 portland cement	100
Sand	174
Aggregate (0.75 in. max.)	279
Water	41

Test results on the concrete were as follows:

Water/cement ratio = 0.405
Air content = 1.8%
Unit weight = 146.9 lb/ft.^{3}
Slump = 2⅝ in.
Compressive strength:
  (1) After steam cure—4,300 psi
  (2) After 28 days—6,300 psi

The prestressing strand was K270. Two strands were used: one near the upper (compressive) surface of the beam, and one near the lower (tension) surface. The upper strand was necessary to prevent camber on release of the pretensioning load after curing.

The pretensioning strand was positioned by holes in the respective bulkheads to give the nominal concrete covers given in Table F-1. Because the forms were placed in line in the prestressing bed, each beam received the same pretensioning. The applied load, which was the same on each strand, was such as to give a stress of 188,500 psi in the prestressing steel.

Prior to casting, ⅛-in.-diameter cotton twine was wrapped around the lower strand at three points near the beam center on 8 of the 16 beams. The twine was quite absorbent; hence, it was hoped that these areas would act as simulated voids between the steel and concrete during later corrosion testing. One "void" was placed on the lower strand at the beam center, one 3 in. to the right of center, and one 3 in. to the left. Three turns of the twine were made before tying tightly, thus providing a "void" approximately ⅛-in. thick by ⅜-in. long around the prestressing steel. In Table F-1, all beams that contain such "voids" are identified by the letter "V" following the beam number.

It would, of course, have been preferable to form the beams with true voids. However, no satisfactory method could be devised to accomplish this on a production run; therefore, the cotton twine method was used.

All beams prepared for this study were cast from the same batch of concrete. Also, all 16 were vibrated using the customary procedures of the manufacturer (hand-held vibrators). Casting was done in the 4-in. direction.

After a 4-hour preset, the beams were steam-cured for 12 hours, at which time the concrete reached a temperature of 155 to 160°F.

The sides of the plywood forms were removed after the steam curing. Resistance-wire strain gauges were then epoxied to the tension side of four of the beams near their midpoint. On the following day, strain readings were obtained on these four gauges, after which the stress in the pretensioning strand was released and second readings were obtained. The difference in the two readings was the compressive strain in the concrete caused by the pretensioning. These strain values were used later to determine the midpoint loading required to balance the maximum fiber stress (compressive) in the concrete.

BEAM LOADING

The beams were loaded in pairs using the arrangement shown in Figure F-1. The two beams of each pair had the same live load and the same concrete cover—the only difference in the two being that one beam was free of simulated voids, whereas the second contained the three simulated void areas mentioned earlier.

The design of the load jack is shown in Figure F-1. The jacks were calibrated at a fixed position of the adjustment nut and they were then used for loading the beams while in this same orientation. Load was applied to the two beams simultaneously by tightening the ⅜-in. nuts at the end clamps or cradles. The procedure was to apply 250 lb by turning the nuts first at one end and then at the other until the desired load level was achieved. Alignment of the jacks was found to be critical, and it was essential that the center steel bearing plates embedded in the two beams be parallel to within a few degrees. This was accomplished by shims placed under the beams at the wooden supports.

Load-deflection curves were obtained for several beams in which cracking of the concrete was desired. Figure F-2 shows the upper region of the curve obtained for the midpoint deflection of beam B-2V. The midpoint deflection values for these curves were obtained by stretching a 5-mil-diameter tungsten wire across the 5-ft 4-in. span and then measuring with a microscope the movement of the wire with respect to a fixed point at the midspan of the beam.

A load of 2,000 lb was required to relieve all of the compressive stress in the outer fibers of beam B-2V. At a load of 2,500 lb the change in slope of the loading curve indicates that microcracking has occurred in the concrete. A careful inspection of the beam surface at this loading with a microscope failed to reveal any cracks. In fact, no cracks could be detected until a load of 3,500 lb was reached. At this load, visible cracks appeared in both beams. The maximum width of the center crack was 0.004 in. in both cases.

The tensile stress in the concrete at which the first microcracks occurred was approximately 400 psi. If the concrete had continued to be crack-free, the tensile stress at visible cracking would have been 1,000 psi.

Figure F-3 shows the curve obtained when the strain from the SR-4 gauge cemented to the tension side of beam B-1 near midspan was plotted against load at midspan. The first microcracking is indicated at the same load as for the midspan deflection (i.e., 2,500 lb).

Similar tests made with other beams showed that the load required to generate the first microcracks varied with concrete cover, as indicated by the following summary:

BEAM NO.	CONCRETE COVER (IN.)	MIDSPAN LOAD TO GENERATE FIRST MICROCRACKS (LB)
B-11	0.5	2,800
B-1	0.75	2,500
B-15	2.0	2,250

TABLE F-1

SUMMARY OF PRETENSIONED BEAM TESTS

BEAM NUMBER	NO. OF SIMULATED VOIDS[a]	CONCRETE COVER		MAX. FIBER STRESS IN CONCRETE[b] (psi)	MAX DEPTH OF OPEN PORES (in.)	EFFECTIVE CONCRETE COVER[c] (in.)	NO. OF CRACKS IN CONCRETE IN TEST AREA	MAX CRACK WIDTH (in.)	CORROSION OF STEEL STRAND			
		NOMINAL (in.)	MEASURED (in.)						CORROSION INDEX[g]	NO. OF WIRES CORRODED	MAX. PIT DEPTH (in)	EST. RED IN STRAND AREA[h]
B-1	0	3/4	11/16	+1000	1/8	9/16	1	0.008[e]	4	7	0.052	15
B-2V	3	3/4	3/4	+1000	3/16	9/16	1	0.008[e]	4	7	0 054	25
B-3	0	3/4	3/4	0	1/8	5/8	0	0	2	2	[k]	0
B-4V	3	3/4	13/16	0	3/16	5/8	0	0	3	2	[k]	0
B-5	0	3/4	3/4	+250	1/4	1/2	0	0	2	2	[k]	0
B-6V	3	3/4	3/4	+250	1/8	5/8	0	0	2	2	[k]	0
B-7	0	3/4	3/4	-1300	1/8	5/8	0	0	2	2	[k]	0
B-8V	3	3/4	5/8	-1300	1/4	3/8	0	0	4	3	0 014	1
B-9	0	3/4	11/16	+500	1/8	9/16	0	0	3	2	[k]	0
B-10V	3	3/4	5/8	+500	1/8	1/2	0	0	3	3	[k]	0
B-11	0	1/2	9/16	0	5/16	1/4	1[d]	[f]	3	3	[k]	0
B-12V	3	1/2	5/8	0	1/8	1/2	0	0	2	2	[k]	0
B-13	0	1-1/2	1-1/2	0	1/8	1-3/8	0	0	1	0	0	0
B-14V	3	1-1/2	1-1/2	0	3/8	1-1/8	0	0	1	0	0	0
B-15	0	2	2	0	0	2	1[d]	[f]	1	0	0	0
B-16V	3	2	2-3/16	0	0	2-3/16	0	0	1	0	0	0

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[a] Voids formed by cotton string tied around strand, one opposite center of 3-in.-diam. test area and the other two 3 in to either side of center.

[b] Minus value signifies compression. Plus values are computed tensile stresses in concrete if no cracks were present.

[c] Measured cover minus the maximum depth of open pores in test area.

[d] Crack 0.004 to 0.006 in. wide induced by overload after which load level was reduced to give maximum fiber stress as indicated

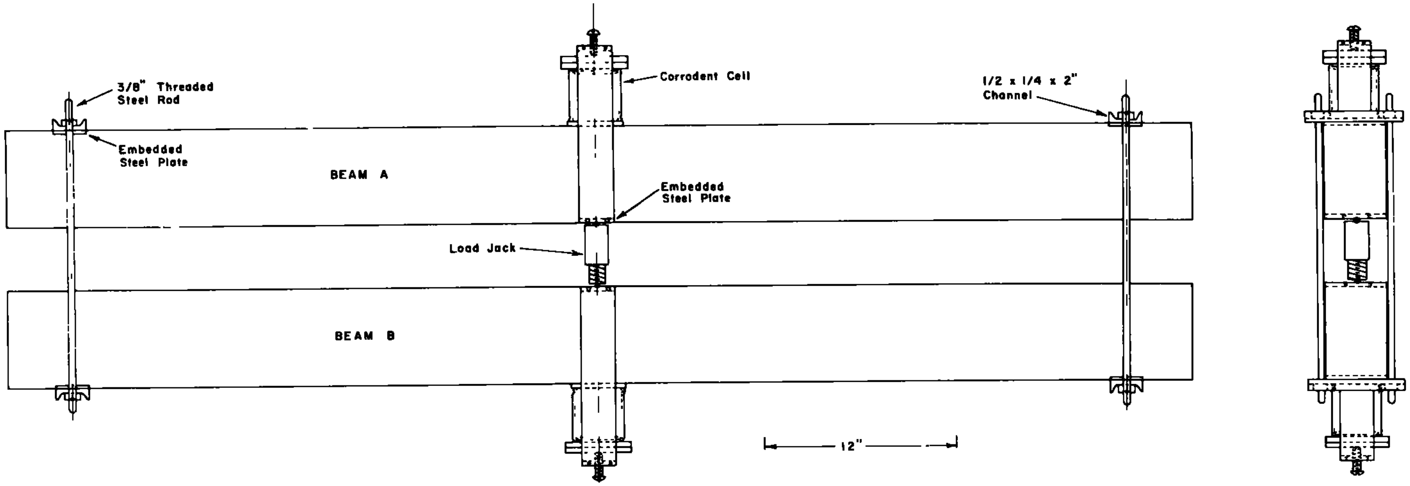
[e] Measured after ten months of testing

[f] No crack detectable after reduction of load Crack had partially rehealed after ten-month exposure

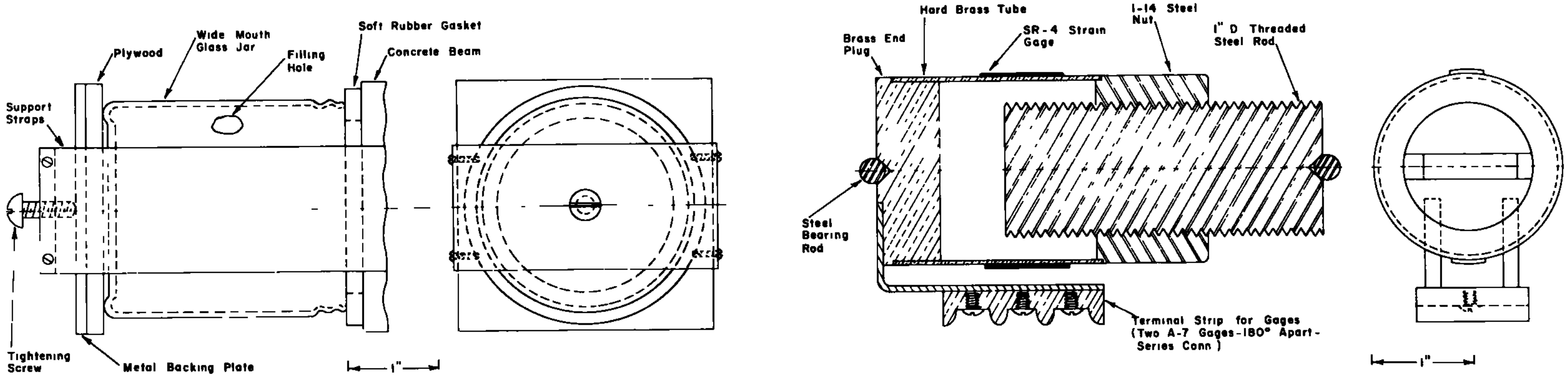
[g] Arbitrary index, 1 - no corrosion detected, 2 - very slight rust staining on adjacent concrete, 3 - rust stains on concrete and steel but no measurable pitting, 4 - pitting of steel

[h] From estimated reduction in cross-sectional area of strand at location of maximum corrosion

[k] No detectable pitting.



(A) DUAL BEAM TEST ARRANGEMENT



(B) CORRODENT CELL

(C) LOAD JACK

Figure F-1. Testing arrangement for pretensioned beams.

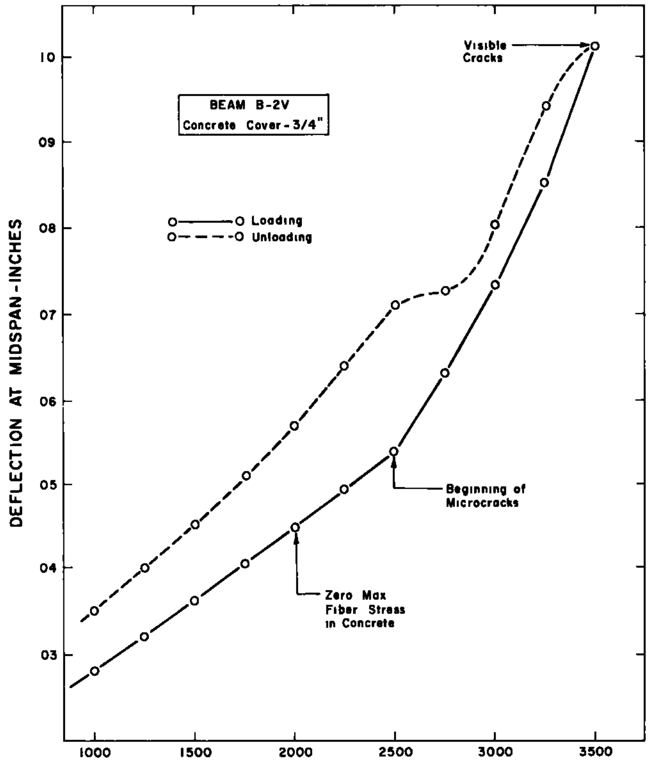


Figure F-2. Upper region of load-deflection curve for pretensioned beam B-2V.

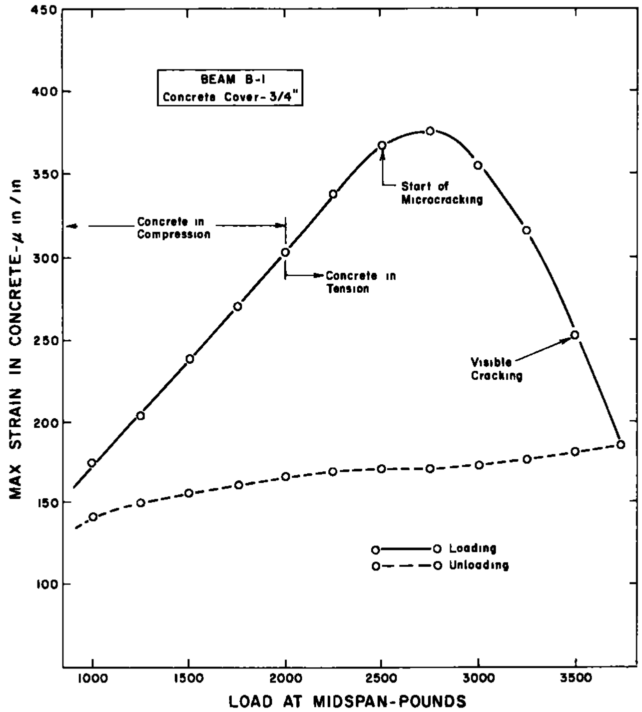


Figure F-3. Upper region of stress-strain curve for beam B-1. Strain was obtained from SR-4 gauge cemented to tension side of beam near the midspan.

Table F-1 gives the applied stresses for the 16 beams. The variables mentioned earlier are all incorporated. It should be emphasized that with the exception of B-11 and B-15 (which were introduced into the test to determine the effect of an accidental overload) none of the other beams was at any time subjected to a higher stress level than that given in Table F-1.

BEAM TESTING

Immediately after loading, the corrodent cells shown in Figure F-1 were attached to the tension side of the beam at midspan and filled with 400 ml of a 3.5% by weight solution of sodium chloride.

Beams B-1 and B-2V, which were heavily loaded, both contained a 0.004-in. crack in the concrete at the area where the cell was to be attached. To prevent leakage, the portion of the crack beneath the cell was sealed with epoxy cement prior to filling; the remainder of the crack, however, was left open to the air.

The soft rubber gaskets proved highly effective in providing a seal between the mouth of the glass jar and the concrete. With the exception of B-1 and B-2V, which developed small leaks at the cracks after about 7 months, no troubles were encountered with leaking at the seals. The solutions were replenished weekly. The amounts required each week varied from 10 to 30 ml. Most of this loss occurred from movement of the solution through the concrete laterally to emerge on the outside of the gasket where it evaporated.

Figure F-4 shows a partial view of the completed installation with the corrodent cells in place and filled with salt solution. The temperature of the testing room varied between 60° and 71°F during the 10-month exposure period.

LOAD READJUSTMENT DURING TESTING

Once each week the load on the jacks was monitored and the nuts on the end clamps were tightened to bring the load on the pair of beams back to the initial value. The following summarizes the average rate of load relaxation with time for beams B-5 and B-6V:

TIME PERIOD FOLLOWING INITIAL LOADING (DAYS)	AVERAGE RATE OF LOAD RELAXATION	
	LB/DAY	%/DAY
0-1	400	17.0
1-8	64	2.7
8-15	32	1.4
15-22	14	0.6
22-85	9	0.4
85-286	5	0.2

This tabulation shows that the load relaxation was fairly rapid immediately after loading. This was probably caused

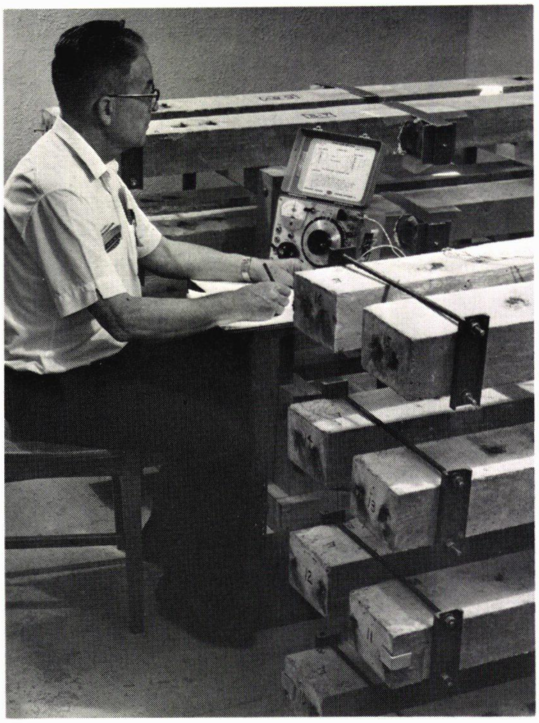


Figure F-4. Partial view of pretensioned test beams.

mostly by creep in the concrete. With incresing time this creep became less and less, but, in these particular tests, there was still a measurable amount of load relaxation, even after 10 months.

The other beam pairs showed similar behavior to B-5 and B-6V. In the case of B-1 and B-2V, the periodic readjustment to maintain a constant load opened the cracks at midspan from a width of 0.004 in. immediately after loading to 0.008 in. after 286 days.

TEST RESULTS

All 16 beams were sectioned with diamond saws after 10 months of exposure to the test conditions. Figure F-5 shows a typical sawed-out section before final removal of the steel.

The results of the examination are summarized in Table F-1. As indicated in the table, the steel was found to be corroded in 12 of the 16 beams. In three of the beams (B-1, B-2V, and B-8V) this corrosion had reached serious proportions, whereas in the other nine the corrosion was still in the very early stages. The four beams with the heaviest concrete cover (1.5 and 2 in.) showed no signs of steel corrosion. Examination of the strand from all beams showed that cement paste had penetrated into most of the interstices during fabrication, thereby providing protection for the center wire.

A discussion of the effect of each of the variables follows.

Effect of Open Cracks

As might be expected, the beams that showed the greatest amount of steel corrosion were the two that were purposely overloaded to generate open cracks in the test area. In the case of B-1, which contained no simulated voids, the corrosion product diffused outward through the crack to form the tubercle shown in Figure F-6.

The greatest amount of corrosion occurred with beam B-1 on that part of the steel strand immediately beneath the crack. The width of the crack on the concrete surface at the time of test termination was 0.008 in. It was observed that even though the crack appeared to be completely filled with corrosion product, corrosion under the crack was apparently still progressing at a fast rate at the time of test termination.

Maximum pitting of the steel occurred on those wires facing the corrodent cell, although all wires in the strand (including the center wire) showed some pitting.

One of the other cracks in beam B-1 was located about 1 in. away from the outer edge of the test area. The width

Figure F-5. Sawed-out center section of beam B-8V after exposure for 10 months. Simulated voids (cotton twine) at each side of center are free of rust staining; center void is darkened by corrosion product. NaCl solution was in a corrodent cell opposite center void.

Figure F-6. Corrosion-product tubercle over open crack in test area of beam B-1.

of this crack at the surface of the concrete was only 0.004 in. at the time of test termination, yet corrosion of the steel was quite pronounced at the point where the crack met the steel. Because the crack was outside of the area encompassed by the corrodent cell, the NaCl solution did not have free access through the crack. Oxygen, however, did have free access, and, because some NaCl solution was undoubtedly present by lateral diffusion through the concrete, all of the necessary conditions were present for electrolytic cell action with an oxygen gradient. This crack was not filled with a gel-like corrosion product, as was the case for the main crack located under the corrodent cell. Filling might have occurred, however, had the exposure time been longer.

The other beam with open cracks (B-2V) contained the simulated voids formed by wrapping cotton twine around the prestressing strand. Maximum corrosion of the steel occurred under the center void. The outer edge of this void was located 0.25 in. away from the point where the center crack met the steel. There was virtually no corrosion observed immediately under the crack; it was concentrated mostly at the simulated void. The crack remained free of corrosion product throughout the testing. In this case, the corrosion product, rather than exuding out through the crack, completely filled the open spaces of the cotton twine.

A second simulated void, positioned 3 in. from the center void (1.5 in. from the edge of the corrodent cell) was also the site of appreciable corrosion. This void, like the one at the center, was located about 0.25 in. from a crack. Here, also, the corrosion of the strand occurred at the void and not at the point where the crack met the steel. This particular crack had a width of 0.004 in. at the concrete surface at the time of test termination. The steel under the third void, which was over 1 in. away from the closest crack, showed no evidence of corrosion.

Effect of Closed Cracks

Beams B-11 and B-15 were purposely overstressed for a period of 10 min at a load level sufficient to form 0.004-in.-wide cracks at the tension surface of the beam. The load was then released and the beams were later loaded to a zero maximum fiber stress for the duration of testing. Although some corrosion of the steel occurred on the beam with the 0.5-in. cover (B-11) the corrosion sites were not associated with the crack positions. In addition, an appreciable flexural force was required to break the 0.5-in.-thick slab similar to the one shown in Figure F-5, suggesting that there had been some rehealing of the crack during the 10-month testing period.

No steel corrosion of any kind occurred in the case of the other precracked beam (B-15), which had a 2-in. rather than a 0.5-in. cover.

Effect of Concrete Cover

As given in Table F-1, the nominal concrete covers over the strand varied from 0.5 in. to 2 in., and the measured covers did not deviate widely from those desired. However, all beams with covers less than 2 in. contained open pores on the test surface which, in effect, decreased the concrete cover over the strand. The beams were cast with the test surfaces at one side of the form. When the strand was close to the form wall (0.5 and 0.75 in. cover) it was apparently difficult for the operator to suitably compact the concrete that was to later form the concrete cover for the steel. In the case of the heavier covers, however, it was possible for the operator to place the vibrator tool between the strand and the form wall and thereby obtain a more suitable compaction.

A second effect of this difficulty of obtaining suitable compaction for the thin covers was reflected in a greater number of voids in the concrete adjacent to the steel. The number of voids with diameters greater than ⅛ in. were counted along the 3-in.-long test area of each beam. The average number varied as follows:

CONCRETE COVER (IN.)	AVERAGE NO. OF VOIDS
2.0	0
1.5	0.5
0.75	1.0
0.5	1.5

Voids of this type were much less common on the opposite side of the strand (i.e., the side that could be reached more easily with the vibrator tool during compaction of the concrete).

The degree of steel corrosion observed for each beam is given in Table F-1. With the exception of beams B-1 and B-2V, which had open cracks, the only other beam showing serious corrosion after the 10-month testing period was B-8V. The effective cover, because of open pores on the area to which the corrodent cell was attached, was only ⅜ in. This small effective cover apparently permitted NaCl and O_2 to permeate to the steel during the 10-month exposure period. Corrosion was concentrated at the simulated void at the center of the test area, although it had also progressed laterally along the strand for a distance of 1.25 in. from the void in one direction, and 2.25 in. from the void in the other. Measurable pitting of the steel, however, was confined to the simulated void location.

Beam B-11 which had even a smaller effective cover than B-8V (0.25 in. rather than ⅜ in.) did not show the same serious corrosion at 10 months. Beam B-11 was free of the simulated voids. Nevertheless, it is believed that main cause for the difference in corrosion behavior between these two beams was that the concrete cover over B-8V was for some reason more permeable to the salt solution and to oxygen.

It is significant that the four beams with the 1.5- and 2-in. concrete covers showed no corrosion of the steel after 10 months. On the other hand, all beams with covers of 0.75 in. or less showed at least some corrosion.

Effect of Live Loads

The only effect of live loads on steel corrosion in the present testing was when the load level was sufficiently high to generate open cracks in the concrete. These results at least suggest that there is no important difference in concrete permeability to salt solution or to oxygen whether the concrete is under 1,300 psi compression or 500 psi tension.

Effect of Voids

Whenever corrosion occurred in beams with the cotton-twine voids it tended to be concentrated at the voids. However, a word of caution is needed when interpreting these results, because a void formed by wrapping cotton twine around strand is not the same as an open cavity. With a cotton-twine void, the NaCl solution reaching the void wall could be transported to the steel by wick action of the cotton fibers. In the case of an open cavity, this wick action would not be present. In fact, a valid question could be raised as to whether salt solution reaching open cavity walls by permeation through the concrete would ever enter the cavity itself. Small-scale laboratory tests (Appendix E) showed that open cavities of this type did not fill with salt solution during long-time testing. Nevertheless, even though the cavities were not filled with solution, their presence could still have an adverse effect on steel corrosion by serving as sites for high oxygen concentration at the steel and thereby permitting oxygen concentration gradient cells to develop. Some evidence for such an effect was noted for the beams that contained no simulated voids. It was observed that when small voids were present between the steel and concrete, corrosion of the steel tended to be more pronounced. This was not always the case, however, because an occasional void in a corroded area would be free of corrosion product.

SUMMARY OF BEAM TESTS

The results of the beam tests can be summarized as follows:

1. The most serious corrosion of steel strand occurred with the two beams that had open cracks.
2. Cracks in two beams caused by brief overloading followed by lowering of the load to normal levels tended to reheal after 10 months. There was no increased steel corrosion because of the cracks.
3. No correlation was found between applied load level and the amount of steel corrosion, except for those cases where the load level was sufficient to cause open cracks in the concrete.
4. No steel corrosion was observed after 10 months of testing when the concrete cover was 1.5 in. or greater, but it was found in all cases where the cover was 0.75 in. or less.
5. Corrosion of the steel was more pronounced when simulated voids were present between the steel and concrete, but the corrosion occurred only when conditions were such as to permit the salt solution to reach the void.
6. The number of open surface pores (and also of small pores at the concrete-steel interface) was less for the beams with a 2-in. cover than for those with 1.5, 0.75, and 0.5 in. The number of interface pores for the beams used in this test was found to be inversely related to the thickness of cover.
7. When corrosion of steel occurred, it was most pronounced on the part of the strand closest to the source of the salt solution. Cement paste present in the interstices of the strand protected the center wire, except in those cases where open cracks were present in the concrete.
8. Serious corrosion as evidenced by pitting of the steel occurred at one crack in one beam where the crack width was only 0.004 in. at the concrete surface.

APPENDIX G

GROUTING STUDIES

Post-tensioned girders for highway bridges are almost invariably grouted. The grout has three important functions: (1) it protects the tendons from corrosion, (2) it provides improved structural rigidity, and (3) it provides increased safety in the event of an anchorage failure.

The grout most widely used in post-tensioning systems is a pumpable slurry prepared by mixing portland cement and water, either with or without special proprietary additives. The purpose of the additives is to improve flow properties and to minimize shrinkage.

Although grouts have been used for many years, the literature contains very little information on their flow properties, and especially on the relation between these flow properties and the filling characteristics of grouts in ducts. In particular, little is known about void-forming tendencies and the factors that influence these tendencies.

The Research Laboratories of the California Division of Highways is currently conducting a comprehensive study of grouts for post-tensioning. The study, however, is still in its early stages. In the present investigations, close liaison was maintained with that organization, and their help and advice, especially in the planning stages, is gratefully acknowledged. In effect, the investigation as herein described might be looked on as complementing the California Division of Highways effort.

The goals of the present research were:

1. To determine the manner in which the physical properties of grouts change with such variables as water content, mixing procedure, and additives.
2. To relate the effects of these variables on flow behavior and especially on the tendency of the grout to form voids during pumping.
3. To conduct full-scale grouting experiments to determine if the findings of the laboratory studies were applicable to field conditions.

A fourth objective was to derive a theoretical expression based on fluid mechanics that would permit the pumping pressures and flow rates to be predicted in advance of large-scale grouting operations.

LABORATORY STUDIES

Grouts and Mixing Equipment

All grout mixtures used in the study were prepared from Type II portland cement furnished by the Ideal Cement Company, Denver, Colorado. Analyses of the cement lots from which the test grouts were prepared are given in Table G-1.

The water-cement ratios used for the test grouts were controlled at 4.0, 4.25, 4.50, 4.75, and 5.00 gal per sack (94 lb). A ratio of 4.0 gives little fluidity and a 5.0 ratio gives a high fluidity. The ratios were limited to these values in light of the range of ratios considered to be acceptable by the firms answering the survey of current grouting practices discussed in Appendix J.

Two brands of proprietary additives—"Interplast-C" (marketed by Sika Chemical) and "Intrusion Aid-R" (by Prepakt, Inc.)—were used in the grout tests. Their behavior in the grout is very similar; thus, the discussion that follows is limited mostly to the effects of Intrusion Aid. In most cases, the amount of this additive when used in a grout mixture was 0.5% by weight of the cement.

Figure G-1 shows the two types of mixers used to prepare the sample mixes. During the initial phase of the lab tests a Reynolds C110 mixer (marketed by Soiltest, Inc.) was used. Later, a mixer design in current use at the Research Laboratories of the California Division of Highways was adopted to test the effects of mixing speed. This design consists of a set of thin paddles mounted on a vertical shaft which rotates in an enclosed steel cylinder. The shaft is driven by a drill press with variable speed adjustments.

Laboratory Test Procedures

The tests selected as appropriate measures of grout characteristics were: (1) compressive strength, (2) shrinkage of the grout during curing, and (3) fluidity as measured by viscometers and flow cones.

TABLE G-1

ANALYSES AND PROPERTIES OF TYPE II CEMENT LOTS

CEMENT PROPERTY	TEST LOT NO. 1	2	3
Chemical composition (%)			
SiO_2	22.1	22.0	21.0
Al_2O_3	4.7	4.8	4.9
Fe_2O_3	4.0	4.7	4.9
MgO	1.1	1.0	1.4
SO_3	2.3	2.2	2.1
Ign. loss	0.9	1.3	1.6
Insol. res.	0.2	0.2	0.1
Ca_3S	49.9	49.5	51.9
Ca_3A	5.6	4.7	4.9
Specific surface (Blaine) (cm^2/g)	3,537	3,707	3,474
Autoclave expansion (%)	−0.02	0.06	0.04
Time of setting (hr)			
Initial set (Gilmore)	3.17	2.67	3.58
Final set (Gilmore)	4.92	4.17	5.08
Set (Vicat)	1.92	2.33	3.00
Compressive strength (psi)			
3-day	2,600	2,570	2,330
7-day	3,830	3,650	3,440
Air entrainment (%)	8.9	8.0	8.2

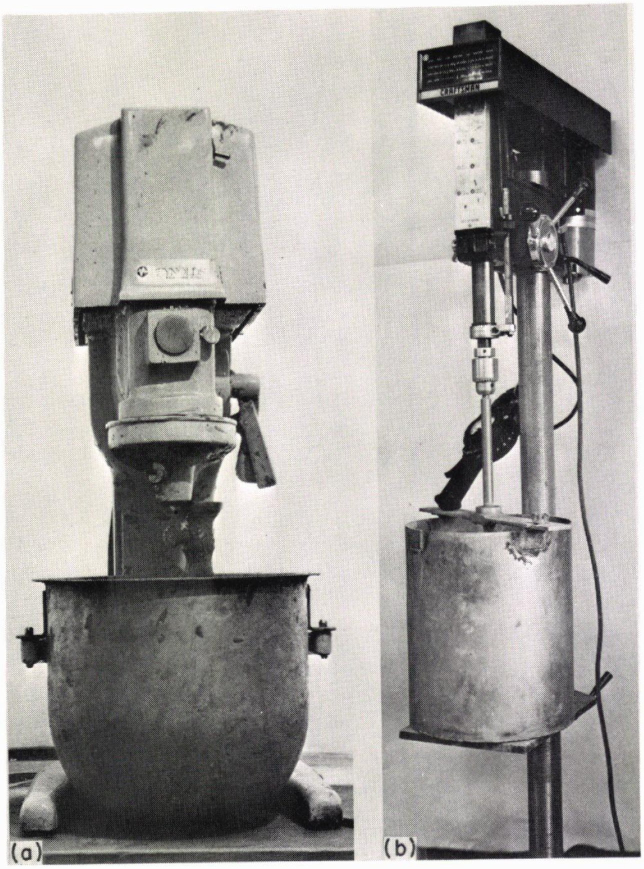


Figure G-1. Types of mixers for grouts prepared in laboratory; (a) Soil Test mixer, (b) Drill-press mixer.

Strength Tests

The strength tests were conducted on a 2-in.-diameter by 4-in.-high samples cast, cured, and tested according to accepted ASTM practice. The samples were cured for time periods of 3, 7, or 28 days and then subjected to ultimate compressive strength tests.

Shrinkage Tests

Shrinkage or expansion of the grout mixtures during curing was determined by measuring the vertical displacement of mixed grout placed in 0.75-in.-diameter plexiglas tubes. The tubes were filled to a height of 10 in. and partially sealed by covering the tops with polyethylene. Shrinkage was determined by measuring the unconfined displacement of the top surface of the grout at time increments of 2 min.

Fluidity

The flow characteristics of the grout were measured using three types of viscosity-measuring devices: a standard flow-cone, a coaxial viscometer designed for the study, and a commercial viscometer.

The flow-cone test was developed by the U.S. Army Corps of Engineers and is described in their specification CRD-C-79-58 (*96*). It was developed as a simple procedure for measuring the relative fluidity of grout mixtures. The test consists of the measurement of the rate of discharge of a grout mixture from the bottom of a standard cone such as is shown in Figure G-2a.

Coaxial viscometers have been used in laboratory tests of fluid viscosity for a number of years. The basic design consists of two coaxial cylinders mounted on a vertical shaft, as shown in Figure G-3.

The grout mixture is placed in the outer cylinder to a height, h. The inner cylinder is closed on the bottom and is raised a distance, d, above the bottom of the outer cylinder. The inner cylinder rotates at a constant angular velocity, ω. The force required to prevent the outer cylinder from rotating is a measure of the fluid characteristics. Figure G-4 shows the viscometer set-up. The outer cylinder is 10 in. high with a 4-in. inside diameter. Three inner cylinders 8.5 in. in height were used. Their outside diameters were 3.0, 3.25, and 3.50 in.

The viscometer was operated by filling the outer cylinder with grout to a depth (h_1) of 0.75 in. with the inner cylinder spacing (d) set at 0.5 in. The inner cylinder was then rotated at angular velocities of 135, 200, 270, and 405 rpm, and the force (F_1) required to prevent rotation of the outer cylinder was measured. Next, the outer cylinder was filled to a depth (h_2) of 8.5 in. The inner cylinder was again rotated as described previously, and the outer cylinder force (F_2) was measured. These force measurements were then related to shear stress (τ) by

$$\tau = \left[\frac{(F_2 - F_1)\cdot K}{\Gamma_1 + \Gamma_0}\right] \div \left[2\pi\left(\frac{\Gamma_1 + \Gamma_0}{2}\right)\cdot(h_2 - h_1)\right] \tag{G-1}$$

This value is an estimate of the shear stress on the grout layer half-way between the outer and inner cylinders. To determine the shear rate ($\dot{\gamma}$), which is a measure of the speed at which the intermediate layers move with respect to each other, the following equation was used:

$$\dot{\gamma} = \frac{2\pi}{60}\cdot\omega\cdot\left(\frac{\Gamma_1}{\Gamma_0 - \Gamma_1}\right) \tag{G-2}$$

Eqs. G-1 and G-2 describe the stress-strain relationships of the fluid. A plot of these values describes a curve of the form

$$\tau = f(\mu\,\dot{\gamma}) \tag{G-3}$$

in which (μ) is the dynamic viscosity of the fluid.

Commercial viscometers are designed to make rapid measurements of fluidity with relatively simple procedures. The viscometers presently available are calibrated for measuring the viscosity of Newtonian fluids. When applied to non-Newtonian fluids they register what is described as an apparent viscosity. However, field testing of grout mixes is generally intended to determine (1) whether the grout is too fluid and thus likely to be subject to excessive shrinkage, and/or inadequate strength, and (2) whether it is too viscous to pump the required duct length without using excessive pressure. Thus, the apparent viscosity readings could be used to develop "experience curves" similar in use to those presently suggested for the flow cone test without being concerned with actual fluid viscosity.

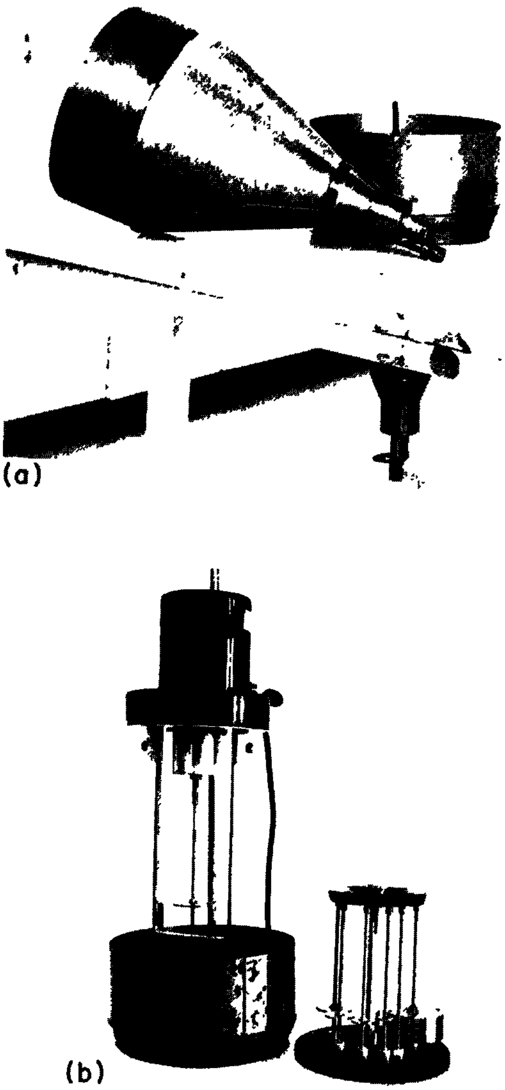


Figure G-2. Commercial fluidity-measuring equipment; (a) Flow cone, (b) Brookfield viscometer.

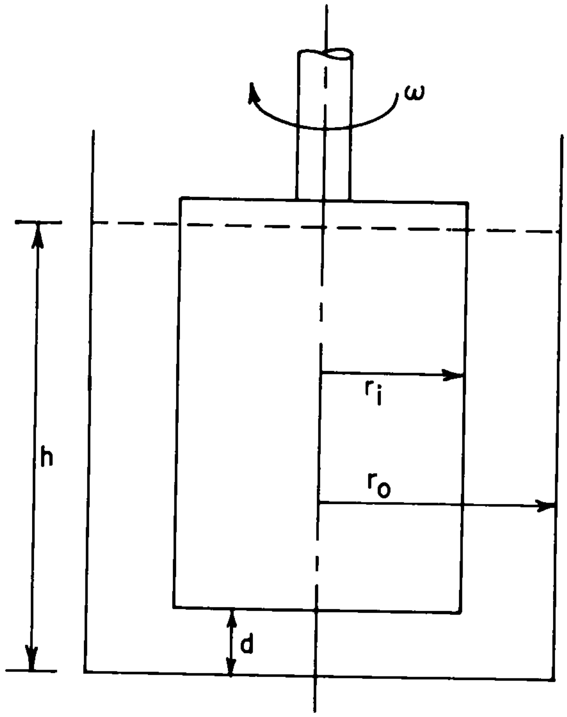


Figure G-3. Coaxial viscometer.

An RV Synchro-Lectric Viscometer, marketed by the Brookfield Company (Fig. G-2b), was used as the commercial instrument. It measures fluidity by means of a rotating spindle submerged in the liquid. The resistance to rotation is registered on a calibrated scale. Seven spindle sizes are provided for liquids of differing viscosities. Eight rotation speeds can be selected, depending on the liquid and the spindle being used. For this study, spindle No. 3 was found to be the most sensitive to variations in grout fluidity.

LABORATORY TEST RESULTS

Effects of Water-Cement Ratio

Compressive strength tests were performed on grout mixtures with 4.0, 4.5, and 5.0 water/cement ratios. After 3 days of curing the strengths were 5,200, 4,500, and 3,000 psi, respectively. The 28-day strengths were 7,500, 6,800, and 5,000 psi.

Typical results of the unconfined shrinkage tests are

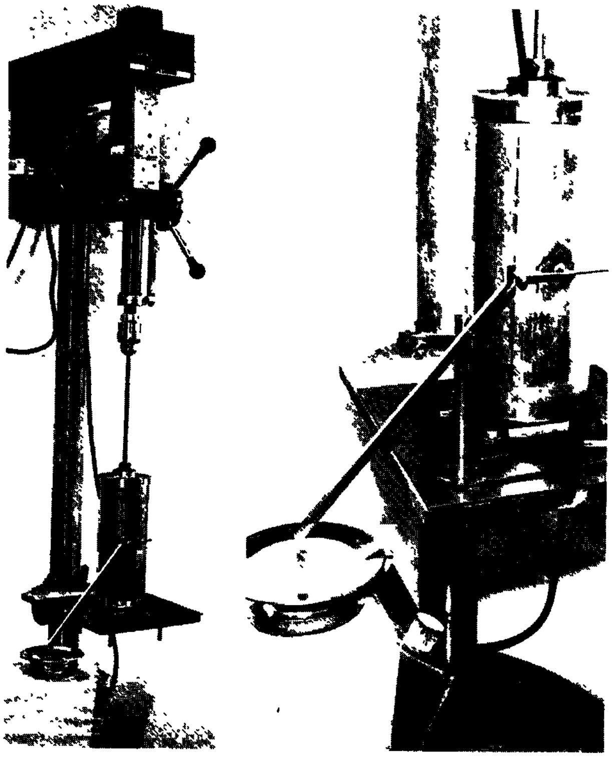


Figure G-4. Laboratory-built coaxial viscometer.

shown in Figure G-5. The total shrinkage of the neat cement mixes after 8 hr varied from 2.2% for the 4.0 mix to 3.8% for the 5.0 mix. Of more significance is the time rate of shrinkage, discussed in the section on additives.

Flow characteristics as exhibited by the flow cone and the viscometers were significantly affected by the w/c ratio. Figure G-6a shows typical variations in flow cone measurements. The mixes with w/c ratios of 4.0, 4.25, and 4.50 exhibited flow characteristics that were significantly lower than the 4.75 and 5.0 mixes at the 540-rpm mixing speed.

The coaxial viscometer measurements in Figure G-6b show that grout has flow characteristics typical of the so-called Bingham plastic type of non-Newtonian fluids. The equations of the stress-strain curves are of the form

$$\tau = \mu \dot{\gamma} + C \qquad \text{(G-4)}$$

in which μ is the apparent dynamic viscosity of the fluid and C is the resistance to flow deformation. The primary effect of changing the w/c ratio was a change in the slope of the curve.

Commercial viscometer measurements are shown in Figure G-6c. The equations of these curves are similar to those of the coaxial viscometer, in that both instruments measure the same stress-strain relationship of the grout. Comparing the 4.75-mix curves, the calibrated coaxial viscometer equation is

$$\tau = 0.006\,\dot{\gamma} + 1.3 \qquad \text{(G-5)}$$

and the commercial viscometer equation for the same mix is

$$R = 1.2S + 24 \qquad \text{(G-6)}$$

in which R is a scaled estimate of the fluidity of the grout when tested at rotation speed, S. Similar equations can also be determined for the other w/c ratio curves.

A comparison of the flow cone data in Figure G-6a and the commercial viscometer data in Figure G-6c suggests that the rapid decrease in flow rate of grout with w/c ratios below 4.75 is reflected by a rapid increase in the apparent viscosity of the grout.

Effects of Mixing Procedure

The compressive strength tests indicated no significant changes in ultimate strength of the hardened grout resulting from changes in mixers, mixing time, or mixing speed. However, grout samples mixed in the drill-press mixer exhibited the more consistent strength values.

In the testing that was conducted, neither the speed nor the time of mixing caused the shrinkage to vary by more than 0.5%.

Figure G-6a shows the effect of mixing speed on the flowability of the grout, as measured by the flow cone. The mixes with w/c ratios in the 4.0 to 4.75 range indicate a significant increase in fluidity with mixing speed, whereas the mixing speed had no significant difference on 5.0 mixes.

Effects of Proprietary Additives

Inclusion of a proprietary additive in the grout mixes resulted in sizeable reductions in their compressive strength. The 3-day strengths were reduced as much as 45%, whereas the 28-day strength tests indicated reductions of as much as 60%. The 28-day tests at 5.0 w/c ratio mixes with additive yielded strengths of as low as 2,000 psi.

The time rate of shrinkage of grout shown in Figure G-5

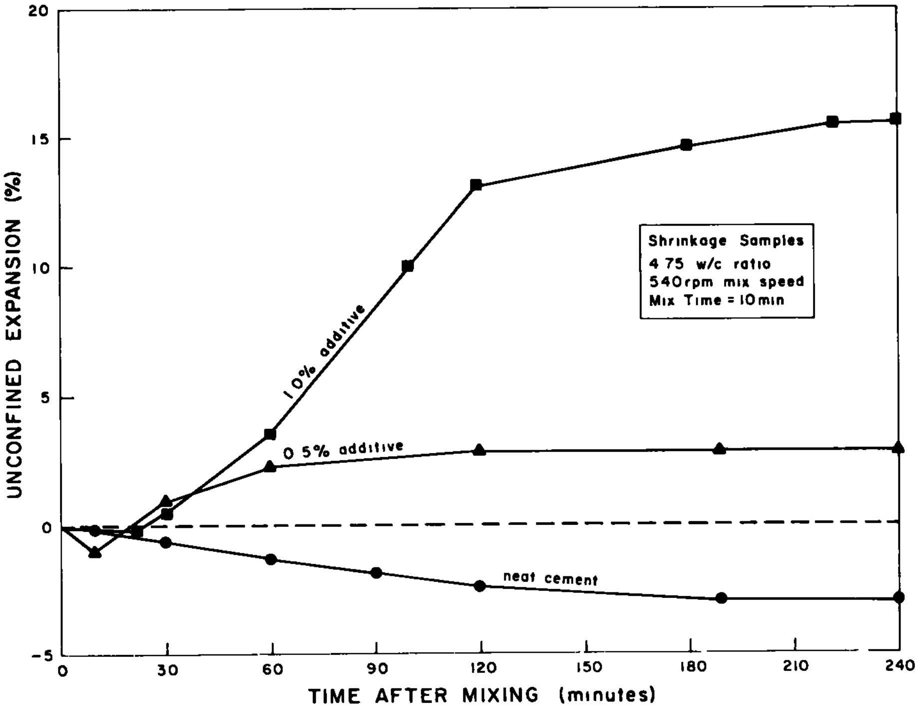


Figure G-5. Dimensional changes of grouts.

indicates the effect of an additive on the shrinkage. A total shrinkage of 2 to 4% was changed to an expansion of 2 to 3% by the introduction of 0.5% additive. This net change was not immediately realized. Figure G-5 shows that during the first half-hour of setting, the additive grout actually incurred a shrinkage greater than that of the neat cement grout. During this early phase of grout curing, the unconfined samples all released free water that rose to the top of the sample tubes. The neat cement samples accumulated water on the order of 2 to 3% by volume of the grout, whereas the samples with 0.5% additive accumulated as much as 5%.

The use of 1%, rather than 0.5%, additive increased the total expansion by a factor of five. However, as in the case of the 0.5% additive grout, the unconfined expansion did not become effective for almost 0.5 hr. Approximately 4% water was released during this period in which the grout was expanding.

When sample tubes containing the additive grouts were pressurized by end caps held in place with a steel clamp the initial shrinkage did not occur and the free water was not released.

The use of an additive in the grout significantly increased its fluidity (Fig. G-7a). Its effect on the 4.50 mix was to reduce the slope (apparent viscosity) of the curve until the slope reached a minimum value approximating that of the 5.0 neat cement grout. Increasing the percentage of additive caused the curve to translate downward, which indicates a reduction of the resistance to flow deformation. This same effect was exhibited by increasing the w/c ratio of grouts with additive, as shown in Figure G-7b.

The grout mixtures with high w/c ratios, and/or additives, exhibited a shear stress that was insensitive to flow velocity. However, when the w/c ratio was decreased below 4.75, the fluidity characteristics appeared to change, indicating that grout flow velocity becomes a critical factor in determining the pumping pressures required for grouting.

The laboratory tests did not provide sufficient data to develop a quantitative correlation between the flow cone and the commercial viscometer. Neither of these instruments is capable of accurately distinguishing differences between a 4.5 grout with 0.5% additive and a 5.0 neat cement grout. The commercial viscometer, however, provides somewhat better data for characterizing these differences because both the slope of the curve and the zero speed intercept are available.

Allowing the grout to set for a "quiescent" time of 20 min before measuring its efflux time provides a second parameter to the flow cone test. However, the extensive settling and segregation of the grout during this quiescent time makes the flow measurements very insensitive to the true pumping characteristics of the grout.

Flow Behavior of Grouts in Plexiglas Models

The flow behavior of grout in transparent "ducts" was studied using two model designs. The grouts were restricted to those having w/c ratios of 4.5, 4.75, and 5.0 gal/sack, both with and without an additive. All mixing was done in the drill-press equipment for 10 min at a mixing speed of 540 rpm.

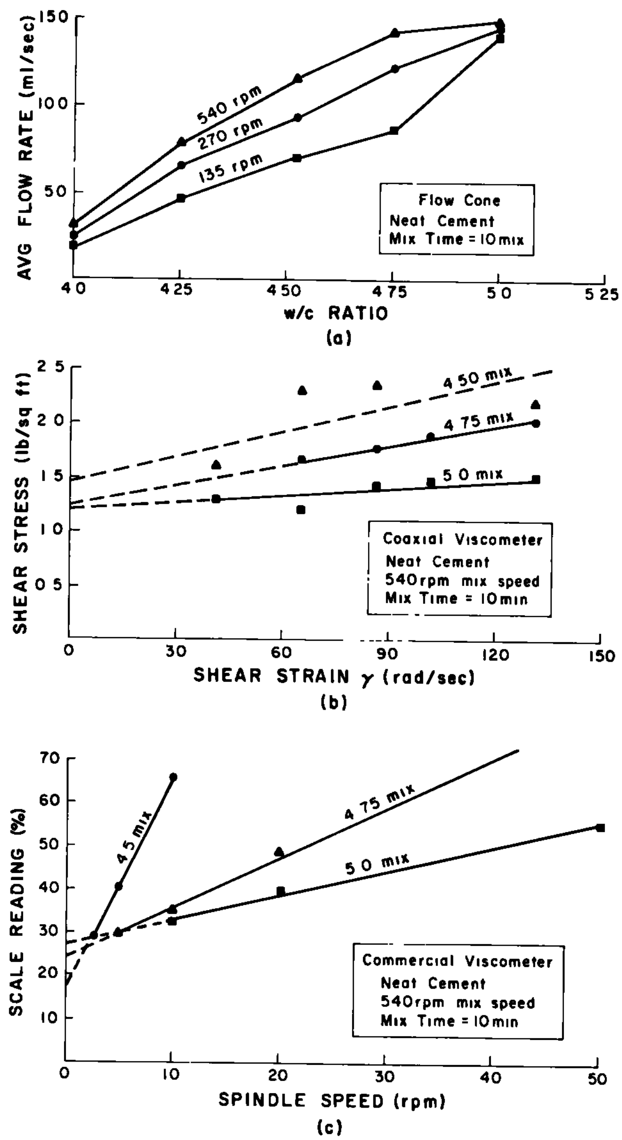


Figure G-6. Fluidity measurements with grouts.

Obstruction Model

The first model was of rectangular cross section, 3 in. high by 1.25 in. wide. The length was 30 in. A total of six plexiglas obstructions with various angles (Fig. G-8) were fastened at intervals along the bottom of the enclosure. The design was such that grouting pressures up to 90 psi could be used without leakage. The walls were constructed with 0.5-in.-thick acrylic plastic (Plexiglas, Type G). End connections were made with standard ⅜-in. pipe fittings.

Tests made with this model with a pressure difference of 2 to 3 psi across the 30-in. length showed a complete absence of void formation. However, when the model was inverted so that the obstacles were at the top, the flow around the obstacles was such as to leave voids up to 1-in. diameter unless the pressure difference across the model was reduced to less than 1 psi. Voids of this type formed only behind those obstacles that had vertical or greater than

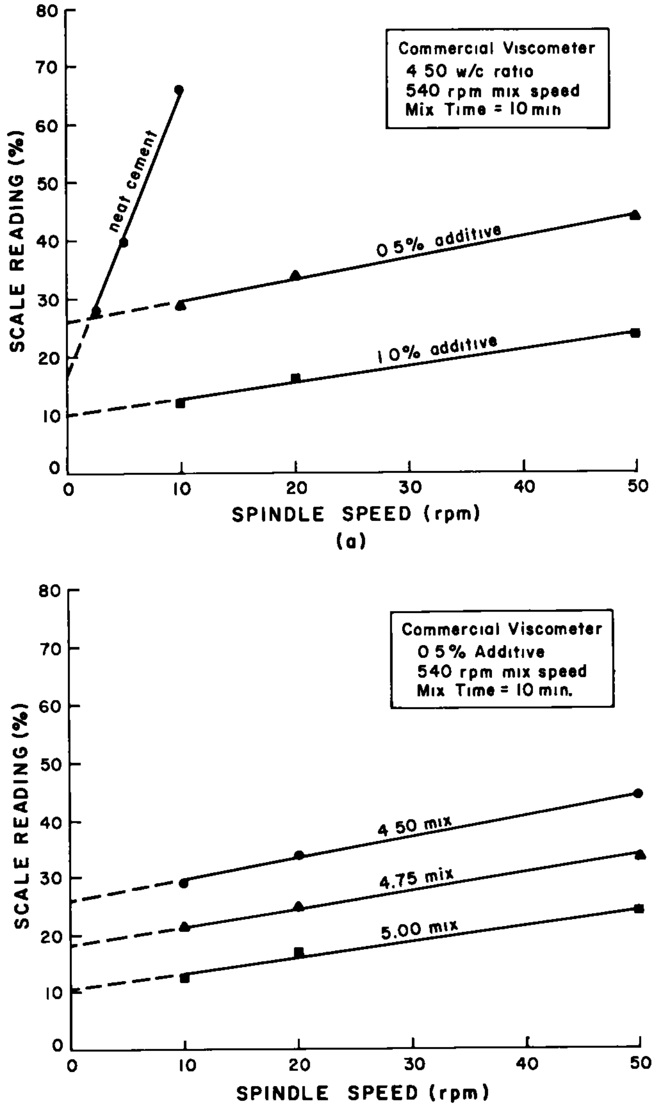


Figure G-7. Effect of additive and w/c ratios on fluidity when measured with commercial viscometer.

vertical angles, as measured from the downstream end of the duct wall.

With the exit valve open so as to simulate standard grouting practice, voids formed readily, even when the obstacles were at the bottom of the model. No voids were formed under these conditions, however, when inlet pressures were below 20 psi, but above 20 psi voids up to 0.75-in. diameter were generated. Higher pressures in these tests resulted in higher flow rates; thus, the mechanism responsible for void formation was undoubtedly high flow rate rather than high pressure.

Stopping and starting the grout in its movement through the model provided information on the increased pressure requirements. When intervals between stopping and starting were 30 sec, it was unnecessary to increase the pressure in order to restart the movement of the grout. When the interval between stopping and starting was 5 min, however, a momentary increase of from 5 to 10 psi was needed to restart the grout, depending on grout viscosity; the mixes with w/c ratio of 4.5 required 10-psi increases in pressure as opposed to 5 psi for the mixes with a w/c ratio of 5.0. When the waiting period was increased from 5 min to 10, 20, and 30 min, a constantly increasing pressure was required to restart the grout flow.

Filling the model with water and allowing the grout to flow through slowly showed that the grout would move the water in front of it until the water was pushed out. The grout mixed with the water along the leading edge only to a limited extent.

One important observation made with the obstruction model was the behavior of the bleed water that was always present on the advancing surface of the grout. This water flowed over the grout by gravity to the low spot on the advancing front. When the grout encountered an obstruction, the water at the bottom of the advancing front tended to be trapped. Grout flowed over the trapped water, thus leaving a small pocket that contained little or no grout. Later, when the grout hardened, this water was reabsorbed into the grout structure, thereby leaving a void.

Formation of voids by this mechanism may be important in some grouting operations. A more important effect of bleed water, however, was observed in the field grouting trials that are described later. In these tests, water formed along the top of the duct by settling of the grout prior to hardening, and this was followed by a migration of the water to the low points in the drape. Later, this water was reabsorbed, thus leaving a thin, river-like void area along the top of the duct.

Cylindrical Models with 0.25-In. Prestressing Wires

Two models were constructed using plexiglas tubes with 0.75-in. inside diameter and ⅛-in.-thick walls. One of the models had a length of 5 ft and the other, 10 ft. Both were prepared so as to have a drape of 1 in. per 5 ft of length. Unstressed wires, 0.25 in. in diameter, were placed in the models to simulate the steel tendons in ducts. End connections to the model were made with standard ⅜-in. pipe fixtures.

Grout pressure was maintained constant at 20 psi throughout all testing by a controlled air pressure over the grout reservoir. This type of pressurizing was found to work satisfactorily, with no evidence of air-bubble generation in the grouts.

Table G-2 summarizes the measurements made during the testing. With 20 psi at the inlet pressure gauge, the average flow velocities varied between 0.63 and 3.53 ft/sec. It was found that the grouts that traveled faster than about 0.9 ft/sec with the exit gate valve at the wide-open position had tendencies to leave void spaces where the wires contacted the inside of the plexiglas tube. No voids were present when the flow was restricted by leaving the exit gate partly closed. When high flow velocities caused voids to form, maintaining the pressure on the grout after the flow had been stopped caused the voids to disappear.

The discharge rates varied from 0.59 to 2.40 gpm, depending both on the number of wires inserted and the w/c ratio of the grout. Figure G-9 is a plot of the flow rates

TABLE G-2

PRESSURES AND FLOW RATES OF GROUTS IN 0.75-IN.-DIAMETER PLEXIGLAS DUCTS

MODEL LENGTH (ft)	GROUT DESCRIPTION W/C (gal./sack)	GROUT DESCRIPTION ADDITIVE[a] (% by wt.)	STEEL DESCRIPTION NO. OF WIRES[b]	STEEL DESCRIPTION S/D AREA RATIO[c]	PRESS. DROP (psi)[d]	FLOW RATE OF GROUT (gal./min)	FLOW RATE OF GROUT (ft/sec)
5	5.00	0	0	0	<1.0	1.09	79
5	5.00	0	0	0	<1.0	1.04	.76
5	5.00	0	1	0.11	1.0	98	.80
5	5.00	0	1	.11	1.0	1.01	.82
5	5.00	0	2	22	2.0	95	88
5	5.00	0	2	22	2.0	.91	.85
5	5.00	0	3	33	3.0	.89	97
5	5.00	0	3	33	3.0	.85	.93
5	5.00	0	3	33	3.0	87	95
5	5.00	0	3	33	3.0	.89	97
5	5.00	0	4	44	4.0	73	.95
5	5.00	0	4	44	4.0	.72	93
5	5.00	0	4	.44	4.0	74	.97
5	5.00	0	4	44	4.0	.74	97
5	5.00	0	5	.56	5.0	65	1.05
5	5.00	0	5	56	4.0	59	.98
5	5.00	0	5	56	4.0	68	1.11
5	5.00	0	5	56	5.0	66	1.08
5	4.75	0.5	3	33	<2.0	2.27	2.48
5	4.75	0.5	3	33	<2.0	2.31	2.51
5	4.75	0.5	4	44	3.0	2.20	2.86
5	4.75	0.5	5	56	4.0	2.17	3.53
5	5.00	0.5	3	33	<2.0	2.40	2.61
5	5.00	0.5	3	33	2.0	2.35	2.64
5	5.00	0	3	33	3.0	1.61	1.75
5	4.75	0	3	33	4.0	0.95	1.03
5	4.75	0	3	33	4.5	0.80	88
5	4.75	0	3	33	4.5	0.83	89
10	5.00	0	3	33	5.0	0.71	75
10	5.00	0	3	.33	4.5	0.74	.81
10	4.75	0	3	.33	6.0	0.58	.63
10	4.75	0	3	33	5.5	0.62	68
10	4.75	0	3	.33	4.0	0.80	87
10	4.75	0.5	3	.33	3.5	0.95	1.04
10	5.00	0.5	3	.33	3.0	1.40	1.53
10	5.00	0.5	3	.33	3.5	1.60	1.74

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[a] Interplast 'C" additive listed in percent by wt. of cement.

[b] 1/4-in.-diam prestressing steel

[c] Ratio of area of steel to area of duct.

[d] Measured across length of duct while inlet pressure was 20 psig.

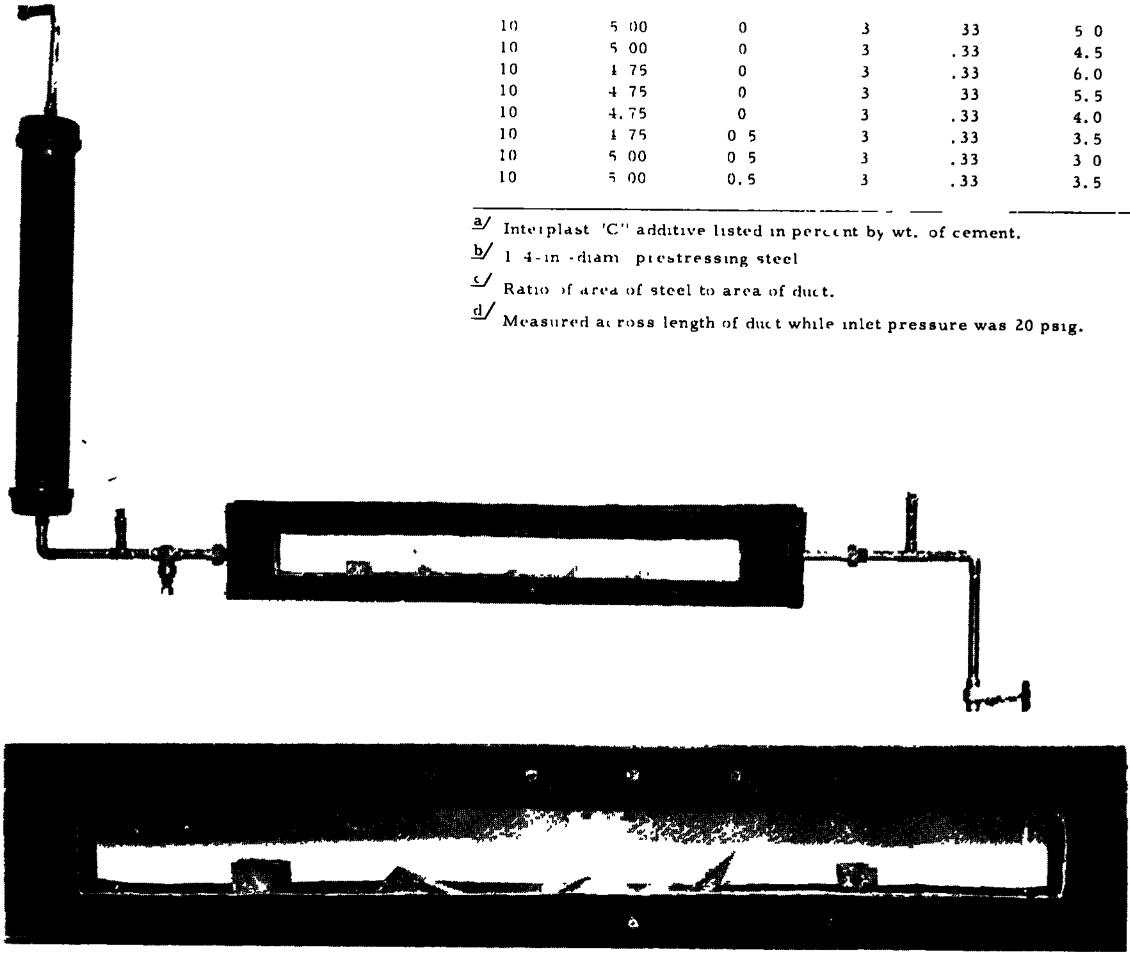

Figure G-8. Plexiglas obstruction-type flow model.

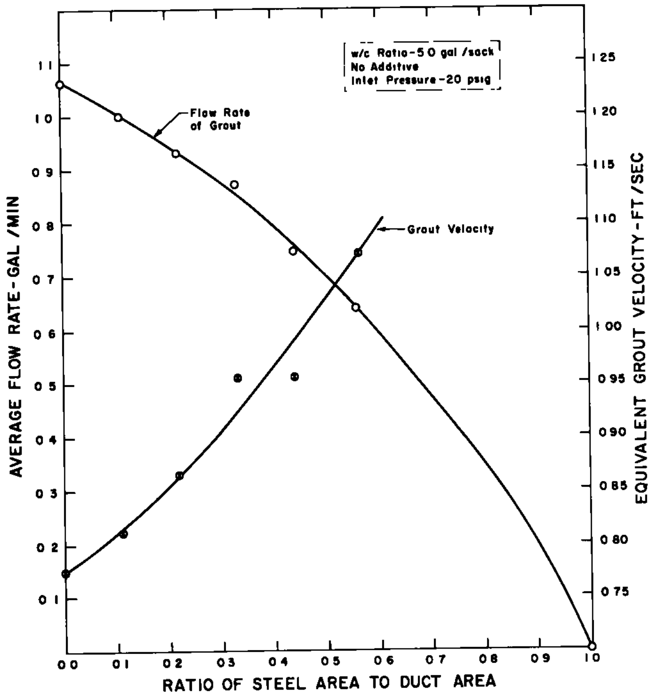

Figure G-9. Flow rates and flow velocities of grout in 0.75-in. I.D. plexiglas "ducts" containing 0.25-in. wires.

observed in the 5-ft model for a grout with no additive having a w/c ratio of 5 gal per sack. Note that whereas the total rate of flow of grout through the model decreases with increasing ratio of steel area to duct area, the velocity at which the grout travels increases. The pressure drop across the 5-ft duct increased about 1 psi for each wire added; for the 10-ft model, the pressure drop was 1.5 to 2 psi for each wire.

Generally, using higher w/c ratio mixes resulted in an increase in the flow rate of the grout. The addition of the additive used in these tests (Interplast-C) gave lower pressure drops and faster flow rates. This was a direct result of the increase in grout fluidity caused by the presence of the additive.

Summary of Model Tests

The more important findings of the tests with plexiglas models were:

1. Voids form more readily at the higher flow velocities.
2. More voids form at high steel-to-duct area ratios.
3. Voids in the grout tend to disappear when grouting pressures are maintained constant after the grouting is completed.
4. Voids can be caused by the presence of bleed water in pockets. This bleed water is reabsorbed after the grout hardens, thus leaving a void in the structure.

Effect of Voids on Rusting of Steel in Grouted Ducts

When a duct is not completely filled, large voids are present between the prestressing steel and the grout. Although the likelihood of voids of this type being filled at some later time with a corrosive solution seems somewhat remote for most post-tensioned bridge girders, the high humidity that would be present in such voids could nevertheless permit steel corrosion to occur. A series of tests was therefore conducted to determine the seriousness of any corrosion that might occur under these high-humidity conditions.

Figure G-10 shows the equipment that was used. A single 0.25-in.-diameter prestressing steel wire 8 in. long was sealed into Pyrex glass tubes with rubber stoppers. Grout

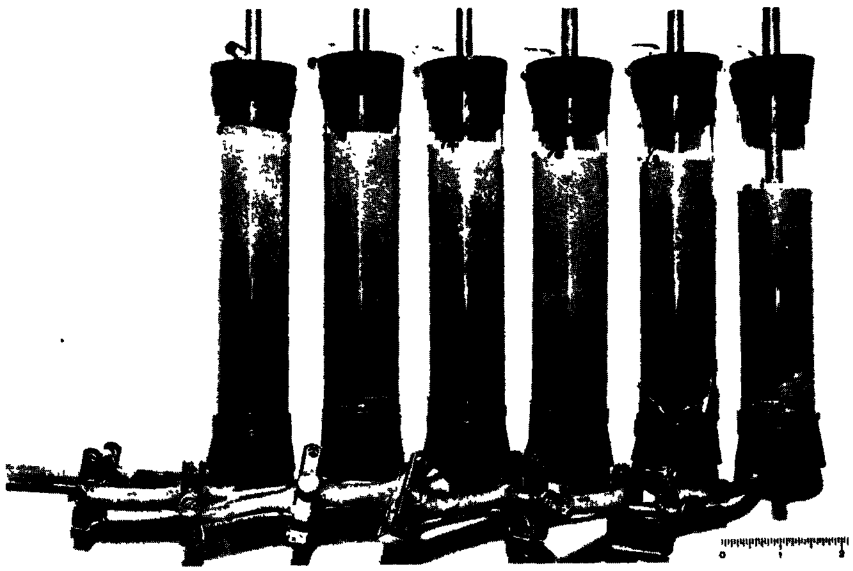

Figure G-10. Test arrangement for investigating rusting of wires in cavities in simulated ducts.

prepared with Type II cement with no additives and with a water/cement ratio of 5.0 was mixed for 10 min at 540 rpm and then introduced into the bottom of each test cell by means of gravity flow. The flow was stopped in each cell when the grout reached a predetermined level. The height of the void space above the grout was 0, ⅛, ¼, ⅜, ½, and 1 in., respectively.

After filling, the vents at the top of each cell were sealed off and the cells were left in position in the rack. Bleed water to a depth of about 3⁄32 in. appeared at the top of each column within a short time after filling. At the same time, condensation moisture appeared on the inner walls of the glass tube above the grout and the same type of condensation probably occurred also on the steel. After 18 hr, the condensation moisture as well as the bleed water had disappeared into the hardened grout, thereby permitting clear observations of the condition of the steel in the cavity. Rust staining was detectable within the first few days. The following summarizes the observations made on the condition of the steel 3 weeks after grouting:

SPEC. NO.	HEIGHT OF SPACE ABOVE GROUT (IN.)	NUMBER OF RUST AREAS ON STEEL IN VOID SPACE[a]
V-1	0	No rusting
V-2	⅛	1
V-3	¼	9
V-4	⅜	20
V-5	½	27
V-6	1	42

[a] Size of areas varied from about 1⁄32- to ¼-in. diameter.

After 3 months the steel was removed and examined. The maximum pit depth under the rusted areas was 1.5 mils. This occurred for the largest void. No corrosion was detected where the steel was in contact with the grout.

These observations show that corrosion of steel can occur in the cavities of an improperly filled duct. Therefore, it is important that ducts be completely filled if such corrosion is to be avoided.

Use of $Ca(OH)_2$ in Flushing Water as a Rust Inhibitor

Occasionally in grouting operations the ducts are flushed with water several days before grouting. This is an undesirable practice from the standpoint of steel corrosion because, during the flushing operation, water is trapped in crevices of the tendons where steel comes into contact with steel. Because this water cannot evaporate (due to the high humidity in the duct), crevice corrosion can then occur.

A few tests were made to determine if flushing the ducts with water saturated with $Ca(OH)_2$ would minimize this type of corrosion. Specimens 6-in. long of both 0.25-in. wires and 0.5-in. strand were used. The 0.25-in. wire specimens were fastened tightly into bundles of three, whereas the specimens of strand were also fastened tightly to insure close contact of the individual wires.

One specimen of each type was dipped into tap water and a second was dipped into water saturated with $Ca(OH)_2$. They were next placed into a closed container with water in the bottom and left undisturbed for a 2-week period. The specimens were then removed and examined for the presence of crevice corrosion. Rusting with some mild pitting of the steel occurred at steel contact points for all specimens that had been dipped in water. In the case of the wire specimens dipped in $Ca(OH)_2$ solution, a few points of mild superficial rusting were observed, whereas for the strand specimens dipped in $Ca(OH)_2$ solution no rusting of any type was detected.

These results suggest that if grouting is to be delayed for more than a few hours after flushing, the flushing should be done with a corrosion inhibitor added to the flushing water. Calcium hydroxide, because of its relatively low cost ($0.50 per lb) and the small amounts needed for saturation (2 lb per 100 gal of water), would appear to be a suitable material for this purpose.

DERIVATION OF RELATIONSHIPS FOR PREDICTING PRESSURES AND FLOW RATES IN DUCTS

Through use of the expressions given here it is possible to obtain values for pumping pressures in ducts of various sizes with different steel-to-duct area ratios. The principal parameter needed for the calculation is the apparent viscosity of the grout as determined with a coaxial viscometer, or its equivalent. The agreement between pressure drops computed by the following expressions and actual pressures observed in long ducts is given in the next section. The derivation is as follows:[*]

The fluid properties of a cement-water mixture is non-Newtonian in nature and can be represented by the general mathematical model

$$\tau = \mu \dot{\gamma} + C \qquad \text{(G-7)}$$

in which

τ = shear stress;
$\dot{\gamma}$ = shear rate;
μ = the viscosity; and
C = the shear resistance to the fluid deformation.

The derivation of the mathematical model for flow of cement grouts in the tendon ducts is based on the assumption that the grout does not change its characteristics during the pumping period and the fluid properties of the grout are independent of time.

After the values of μ and C are determined from tests with a coaxial viscometer, the shear stress may be found from Eq. G-7 if the reasonable value of $\dot{\gamma}$ is known. The total force resistance in a tendon duct of diameter, D, with a length, L, and n tendons of diameter, d, would be

$$F = \pi(D + nd)L\,\tau_w \qquad \text{(G-8)}$$

in which

F = the force;
n = the number of tendons;
L = the length of tendon conduit; and

---

[*] The analysis contained in this section was performed by Dr. Wen H. Huang, Assistant Professor, Department of Civil Engineering, University of Denver

$\tau_w =$ the shear stress at the walls of the tendons and the duct.

The effective pressure loss, in a duct of length L, then, would be

$$\Delta p = C_e \frac{4L\,\tau_w(D + nd)}{D^2 - nd^2} \qquad \text{(G-9)}$$

in which C_e is the effective pressure coefficient depending on the locations as well as the distribution of the tendons in a duct. If the tendons are uniformly distributed in a duct, the value of C_e will be close to 1. The verification of Eq. G-9 is discussed in the section on field tests.

If the relation between shear stress and strain is known, the discharge of the grout in a duct can be derived in the following manner.

The relation between the shear rate, $\dot{\gamma}$, and the shear stress, τ, may be represented in the general mathematical form

$$\dot{\gamma} = f(\tau) \qquad \text{(G-10)}$$

The shear rate, $\dot{\gamma}$, is expressed in the form

$$\dot{\gamma} = \frac{du}{dy} \qquad \text{(G-11)}$$

in which u is the velocity at a point and y is the distance of the point from the wall, as shown in Figure G-11.

For the case of flow of grout in a pipe, the relation $y = R - r$ holds, and its first derivative is $dy = -dr$. The relation in Eq. G-11 now can be represented as

$$\dot{\gamma} = -\frac{du}{dr} \qquad \text{(G-12)}$$

Comparing Eqs. G-10 and G-12, the following relation is obtained:

$$-\frac{du}{dr} = f(\tau) \qquad \text{(G-13)}$$

Referring to Figure G-11, the summation of forces in the Z-direction is represented by

$$\sum F_z = 0 \qquad \text{(G-14)}$$

that is,

$$2\pi r L \tau = \pi r^2 (p + \Delta p - p)$$

and the shear stress found from Eq. G-14 is .

$$\tau = \frac{r \cdot \Delta p}{2L} \qquad \text{(G-15)}$$

in which

$p =$ the unit pressure at the downstream surface of an element;

$\Delta p =$ the difference in unit pressure between the upstream and downstream surface of an element; and

$L =$ the length of the fluid element.

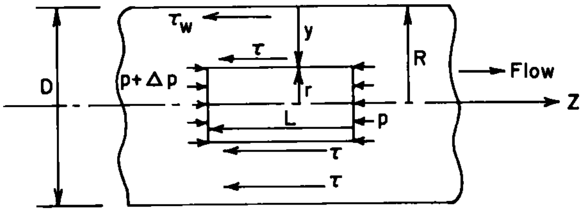

Figure G-11. Forces during flow of grout in pipe of inside diameter, D.

The shear stress at the outer wall is obtained by replacing r with R and is

$$\tau_w = \frac{R \cdot \Delta p}{2L} \qquad \text{(G-16)}$$

If the pressure is assumed to be uniformly distributed, the values of Δp in Eqs. G-15 and G-16 are the same and the relation

$$\tau = \frac{\tau_w\, r}{R} \qquad \text{(G-17)}$$

holds. Eq. G-13, then, can be reduced to

$$-\frac{du}{dr} = f\frac{r\,\tau_w}{R} \qquad \text{(G-18)}$$

Integration of Eq. G-18 gives

$$u(r) = -\int_R^r f\left(\frac{\tau_w\, r}{R}\right) dr = \int_r^R f\left(\frac{\tau_w\, r}{R}\right) dr \qquad \text{(G-19)}$$

and the discharge in a duct, q, is found by the multiplication of velocity and the cross-sectional area; that is,

$$q = \int_0^R u(r)\, 2\pi r dr = \pi \int_0^R u(r)\, d(r^2)$$
$$= \pi[r^2 u(r) - \int r^2\{u(r)\}]_0^R \qquad \text{(G-20)}$$

At $\quad r = 0 \quad r^2 u(r) = 0$
and $\quad r = R \quad u(R) = 0$
therefore, $r^2 u(r) = 0$ at all conditions.

Eq. G-20 can now be reduced to

$$q = -\pi \int_0^R r^2\, du(r) = \pi \int_0^R r^2 f\left(\frac{\tau_w\, r}{R}\right) dr \qquad \text{(G-21)}$$

By rearranging Eq. G-17, the value of r may be calculated from

$$r = \frac{R\tau}{\tau_w} \qquad \text{(G-22)}$$

and Eq. G-22 can be further reduced to

$$q = \frac{\pi R^3}{\tau_w^{\,3}} \int_0^{\tau_w} \tau^2 f(\tau)\, d\tau \qquad \text{(G-23)}$$

The equation of $f(\tau)$ is governed by the characteristics of a fluid. Because the grouts of the cement-water mixture are Bingham plastic-type of fluids as indicated in the mathematical model in Eq. G-7, Eq. G-10 may be written in the form

$$\dot{\gamma} = f(\tau) = \frac{\tau - c}{\mu} \qquad \text{(G-24)}$$

Substitution of Eq. G-24 into Eq. G-23 gives the following result:

$$q = \frac{\pi R^3}{\tau_w{}^3}\int_0^{\tau_w} \tau^2\left(\frac{\tau - c}{\mu}\right)d\tau$$

$$= \frac{\pi R^3}{\mu\tau_w{}^3}\int_0^{\tau_w} \tau^2(\tau - c)\,d\tau$$

$$= \frac{\pi R^3}{\mu\tau_w{}^3}\left[\frac{\tau_w{}^4}{4} - c\,\frac{\tau_w{}^4}{3}\right] \qquad \text{(G-25)}$$

By inserting the relation of Eq. G-16 into Eq. G-25, the following relation is obtained:

$$q = \frac{8\pi L^3}{\Delta p^3\mu}\left[\frac{1}{4}\left(\frac{R\cdot\Delta p}{2L}\right)^4 - \frac{c}{3}\left(\frac{R\cdot\Delta p}{2L}\right)^3\right] \qquad \text{(G-26)}$$

The theoretical discharge rate, q, in a duct of diameter D without tendons can be found from this equation provided that the pressure difference, Δp, and pipe length, L, are known. If the value of Δp is unknown, Eqs. G-7, G-9, and G-26 are solved simultaneously to find the value of q.

When tendons are inserted into a duct, the effective area changes, and so does the discharge in the duct. For simplicity, a short-cut method is used to evaluate the discharge. The average velocity of the grout is found by dividing the discharge shown in Eq. G-26 by the total duct cross-sectional area; that is,

$$\bar{u} = \frac{q}{A} = \frac{32L^3}{\Delta p^3\mu D^2}\left\{\frac{1}{4}\left(\frac{R\cdot\Delta p}{2L}\right)^4 - \frac{c}{3}\left(\frac{R\cdot\Delta p}{2L}\right)^3\right\} \qquad \text{(G-27)}$$

The theoretical discharge for the grout flowing in the duct with n number of tendons inserted would be

$$Q_t = \bar{u}\,A_e = \frac{8\pi L^3}{\Delta p^3\mu}\cdot\frac{D^2 - nd^2}{D^2}\left[\frac{1}{4}\left(\frac{R\cdot\Delta p}{2L}\right)^4 - \frac{c}{3}\left(\frac{R\cdot\Delta p}{2L}\right)\right] \qquad \text{(G-28)}$$

in which

Q_t = the theoretical discharge; and
A_e = the effective area.

Because the tendons in the actual case are not uniformly distributed, the discharge obtained from Eq. G-28 is slightly different from the actual values. If the discharge coefficient, C_d, is introduced, the actual discharge, Q, can be calculated from the following equation:

$$Q = \frac{8\pi L^3 C_d}{\Delta p^3\mu}\cdot\frac{D^2 - nd^2}{D^2}\left[\frac{1}{4}\left(\frac{R\cdot\Delta p}{2L}\right) - \frac{c}{3}\left(\frac{R\cdot\Delta p}{2L}\right)^3\right] \qquad \text{(G-29)}$$

The value of C_d should be determined from the field discharge data.

GROUTING TEST WITH A FULL-SCALE GIRDER

The laboratory studies provided information that could be used for predicting flow behavior of grouts in long ducts. However, because of the difficulty of reproducing all of the conditions that are present during an actual field grouting operation, it was decided to also conduct grouting tests on ducts that were over 300 ft long.

Girder Design and Fabrication

A single girder 107 ft long was employed for the field testing. The girder was purposely designed with a large cross section (2.5 ft wide by 3 ft deep) so as to provide sufficient space to incorporate a total of eight draped tendons.

Figure G-12 shows the positioning of each of the ducts. As shown in the figure, cross-over connections were welded near the ends of six of the ducts, thus providing for two ducts with effective lengths of 321 ft. These connections became an integral part of the beam after the concrete was cast. The remaining two ducts were left open at the ends. Therefore, each of these had an effective length of 107 ft.

Five 107-ft lengths of flexible duct and three 107-ft lengths of rigid duct were incorporated into the design. Both types had nominal inside diameters of 2 in. The slope of the tendon drape was established at 10:1, which was somewhat severe without being impractical. Figure G-12 shows the position of the vents and drains with respect to the drape. The grout inlets for all ducts consisted of ⅜-in. standard pipe connections.

Each duct contained a total of 16 prestressing wires, 0.25 in. in diameter, that had been button-headed at the ends for anchorage attachment. With this arrangement, the ratio of steel-to-duct area was 0.25. The usual practice of the Prescon Corp. in Denver (which furnished the tendons) is to use 16 wires in a 1.75-in. I.D. duct, which computes to an area ratio of 0.33. This practice is used for ducts up to 100 ft long. However, for pumping ducts as long as 321 ft, the slightly lower ratio was believed advisable.

The girder with the tendons in position was cast in a pre-

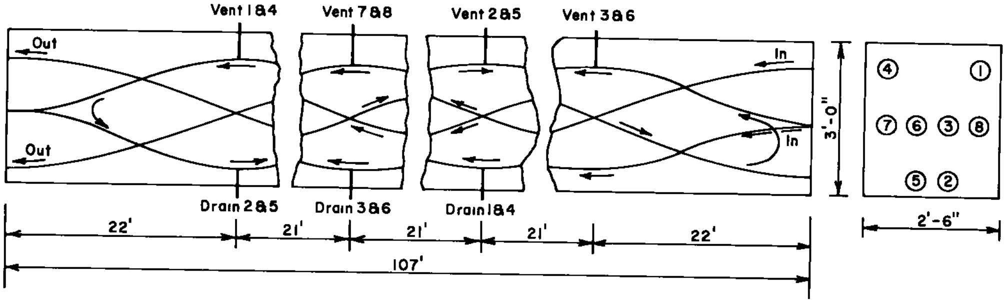

Figure G-12. Schematic of girder.

stressing bed at the Denver plant of Prestressed Concrete of Colorado, Inc. Figure G-13 shows the ducts in the form prior to casting. The girder was cast from a lightweight, low-strength concrete so as to facilitate removal of the tendons for the later examinations. Figure G-14 shows the girder with the forms removed. Figure G-15 is a partial view showing some of the vent valves and pressure gauges.

The wires in the ducts were stressed to 20,000 psi after a 4-day curing period. This stress level was believed sufficient to draw the wires into the same position that they would assume in a fully prestressed system.

Shortly after stressing, each of the ducts was tested for blockages with both compressed air and water. All ducts were found to be free of obstructions. The test water was then drained from the two 321-ft ducts but allowed to remain in the two 107-ft ducts. Figure G-16 shows the stressing end of the girder with the flushing hose attached.

Grouts and Grouting Equipment

Two grouts were selected, based on the results of the laboratory testing. Both were prepared from Type II cement. The water-cement ratio was 4.75 gal/sack. One of the grouts (Grout A) contained no additive, and the other (Grout B) contained Intrusion Aid added as 0.5% by weight of the cement.

The grouts were mixed in a ⅓-hp, 1,725-rpm, twin-paddle commercial mixer. The mixing time of the 25-gal batches for the 107-ft ducts was 4 min; a 15-min mixing time was used for the 100-gal batches needed for the 321-ft ducts.

Pumping of the grout was done with a Moyno 2-hp, screw-type pump. This pump, which was capable of generating grout pressures up to 300 psi, was equipped with a recirculating bypass and a 30-gal reservoir.

Grouting of Ducts

Three days after stressing, the two 107-ft ducts were grouted—one with Grout A and the other with Grout B. The flushing water was allowed to remain in the duct to be pumped with Grout A. The pumping was done slowly by increasing the pressure at the pump to a maximum of 100 psig, with each vent remaining open until homogeneous grout flow was evident. The pressure drop across the length of the duct was approximately 50 psig and the time required for grouting was approximately 2.5 min. The water that had been allowed to remain in the duct was forced out ahead of the grout. When homogeneous grout appeared at the end of the duct, the valve was closed. The pumping pressure was then maintained for several minutes, after which it was released.

The second 107-ft duct was drained of remaining water by air pressure and it was then pumped with Grout B. The pumping procedure was the same as that used for the first duct. The pressure drop across the 107-ft length in this case was 25 rather than 50 psi, but the pumping time was essentially the same as for the first duct. As soon as homogeneous grout appeared at the far end of the duct, the outlet valve was closed and pump pressure was adjusted to 80 psig. The inflow valve was then closed to retain the pressure. However, because of the fabrication clearances required between the wires and the stressing plates, a slight leakage occurred and the pressure dissipated within 20 min.

The first of the 321-ft ducts was pumped with Grout A 6 days after stressing. Standard procedure was followed for the pumping of the first 236 ft, each vent valve being closed when the grout outflow appeared homogeneous. The grout flowed readily during this period, with the grouting pressure never exceeding 150 psig. Through an oversight, pumping of the last 84 ft of the duct was done with the outlet valve closed. The pump pressure mounted rapidly to 260 psig, which was sufficient to cause a "flash set" of the grout, making it impossible to continue the grouting of the final 40 ft of the duct. When the stoppage occurred, the grout inlet valve was immediately closed. Again, as was the case with the 107-ft duct, the pressure at the grout inlet slowly dissipated.

On the next day the remaining 321-ft duct was pumped with Grout B, which was the one containing the additive. During the pumping, the inflow pressure was increased slowly to a maximum of 90 psig. Standard procedures for closing each vent were followed. The time required for grouting the entire 321 ft of duct was approximately 8 min. The inlet pressure at the time the final valve was closed was 80 psig. As in the case of the previous ducts, pressure at the inlet gauge dissipated within ½ hour.

Comparison of Predicted and Measured Pressure Losses in Ducts

The flow times of the 107-ft ducts indicated that the additive had little, if any, effect on grout velocity. The average velocity was 8.5 in./sec. The stress-strain characteristics of the additive grouts with 4.75 w/c ratios suggested a relationship such as

$$\tau = 0.006\,\dot{\gamma} + 0.6 \qquad \text{(G-30)}$$

The shear rate, $\dot{\gamma}$, may be expressed as dv/dy, where $dv =$ 8.5 in./sec. and $dy = 1.0$ in./radian for the 2-in. duct. Thus,

$$\dot{\gamma} = 8.5 \text{ radians/sec} \qquad \text{(G-31)}$$

and

$$\tau_w = 0.006\,(8.5) + 0.6 = 0.65 \text{ psf} \qquad \text{(G-32)}$$

Pressure losses computed with this shearing stress are for a duct without stressing wires. Thus, a corrected shear stress must be determined by calculating an effective pressure coefficient. Using Eq. G-9 and the 25-psig pressure drop of the 107-B duct,

$$C_e = \frac{\Delta p(D^2 - nd^2)}{4L\tau_w(D + nd)} = \frac{25\cdot(4-1)}{4\cdot 107\cdot 12\cdot 0.65/144(2+4)} = 0.55 \qquad \text{(G-33)}$$

If this equation holds true, then the pressure loss for duct 321-B should be given by

$$p = \frac{C_e\cdot 4\cdot L\cdot \tau_w(D + nd)}{D^2 - nd^2} = \frac{0.55\cdot 4\cdot 321\cdot 12\cdot 0.65/144(2+4)}{(4-1)} = 77 \text{ psig} \qquad \text{(G-34)}$$

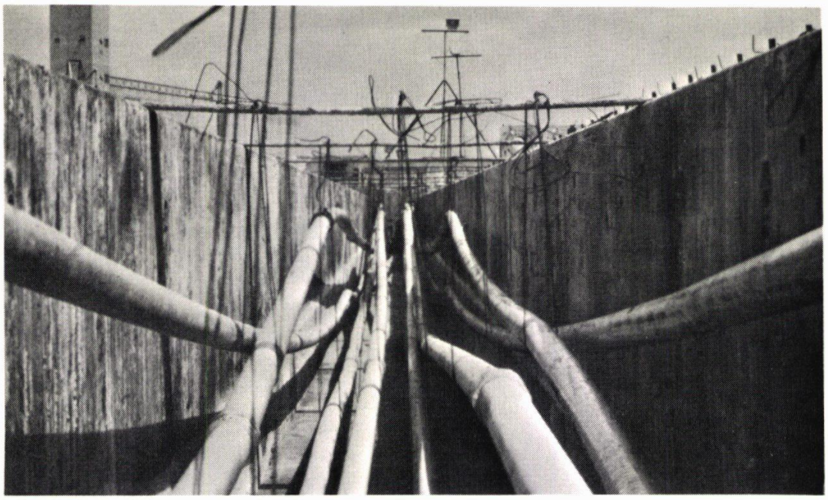

Figure G-13. Casting form for girder, showing positioning of ducts.

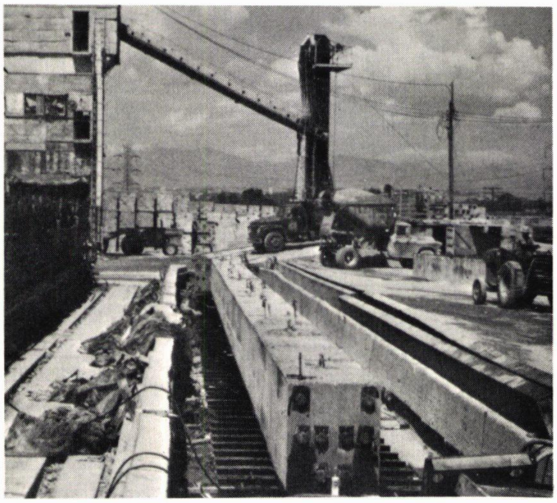

Figure G-14. Girder after removal of forms.

The actual pressure profiles for the test girder are shown in Figure G-17. The loss for duct 321-B was 80 psig.

If the fluid strain rate $\dot{\gamma} = 8.5$ radians/sec. is applied to the experimentally determined curve for the neat cement,

$$\tau_w = 0.006(8.5) + 1.3 = 1.35 \text{ psf} \qquad \text{(G-35)}$$

The corresponding pressure loss for duct 107-A would be

$$p = \frac{0.55 \cdot 4 \cdot 107 \cdot 12 \cdot 1.35/144(2+4)}{(2-1)} = 52 \text{ psig} \qquad \text{(G-36)}$$

Figure G-17 shows that the actual loss for that duct was 50 psig.

No measurements were possible on duct 321-A because of the flash setting of the grout. The predicted pressure loss for that duct is plotted as a dashed line on Figure G-17, with the total loss across the duct being 155 psig. Comparing the pumping pressures prior to the flash setting with duct 107-A suggests that this value is quite reasonable.

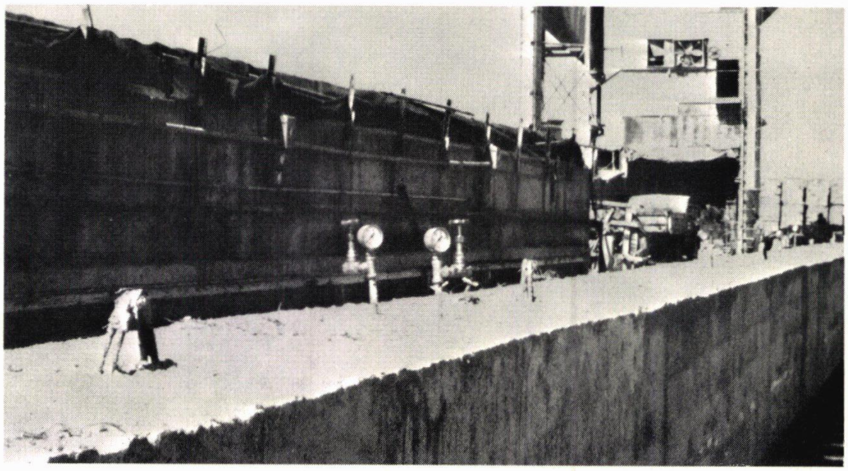

Figure G-15. Partial view of girder, showing vent valves and pressure gauges.

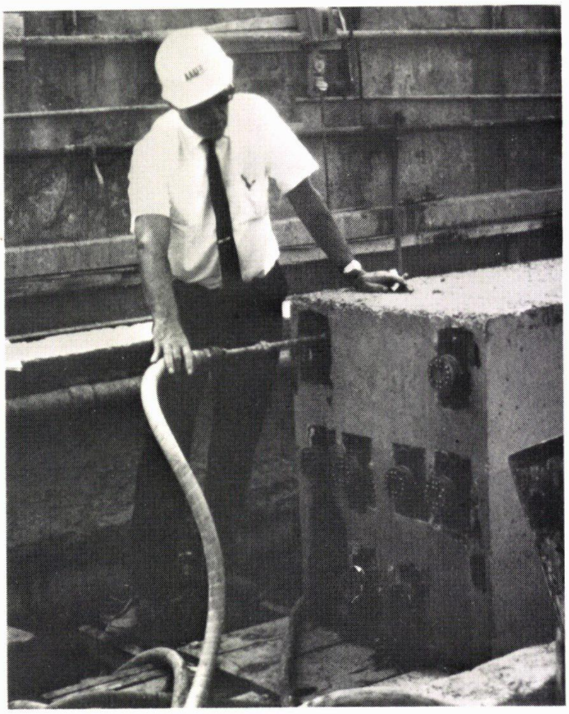

Figure G-16. Stressing end of girder with flushing hose attached to one duct.

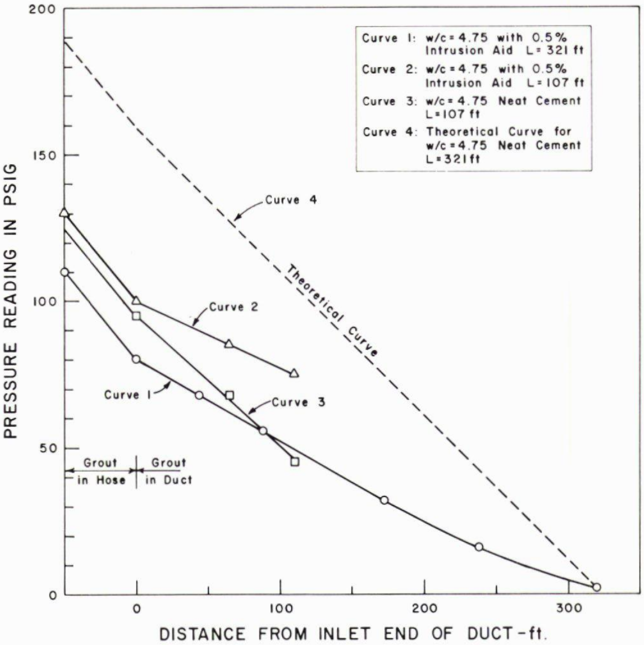

Figure G-17. Pressures in ducts during grouting.

Removal of Grouted-Duct Sections from Girder

Sections of grouted ducts approximately 18 in. long were removed 3 days after the final grouting operation had been completed. Table G-3 gives the locations at which the sections were taken. The selected sample locations included what were considered to be the critical points along the length of the girder.

The sections were removed by cutting through the low-strength concrete as far as the ducts with a jackhammer, followed by a cutting of the tendon with an oxyacetylene cutting torch. The sections thus removed were then returned to the laboratory where they were cut into 6-in. lengths with a diamond saw. Finally, the steel wall of the duct was removed from each of these 6-in. lengths so as to permit a more complete examination of void formation near the walls.

Examinations for Voids

Except for the last 40 ft of duct 321-A (where the flow stoppage occurred because of excessive pressure) all ducts after grouting were free of large unfilled cavities. Small voids, however, were present at numerous locations. The most common were (1) water-track voids (Figs. G-18a and G-18b), (2) internally trapped air voids (Fig. G-18c), and (3) internal voids between the steel and grout (Fig. G-18d). The observed prevalence of each type of void in each sample is given in Table G-3.

As given in the table, many of the samples exhibited water-track voids along the top surface of the duct. It appeared that free water collected and flowed to the low points of the ducts prior to initial setting of the grout. Voiding from water-tracking was evident in both the neat cement grouts and the grouts with additive. Elongated voids of this type were of shallow depth, even the most severe being less than 0.25 in. deep. Their presence would not detract from the corrosion protection of the steel, except possibly at the bottom of a drape where the steel would be in contact with the upper surface of the duct wall, thus permitting the prestressing steel to be present in the void area.

Although the majority of the voids next to the duct wall were associated with water tracks, there were also other void structures that were obviously formed from entrapped air bubbles (Fig. G-18c). Some of these undoubtedly were caused by the flowing grout trapping air pockets at the points where the steel contacted the duct wall. In the case of the rigid ducts the trapped air bubbles were not free to migrate to the top of the duct because of the presence of the wires, and the retained pumping pressure was not adequate to force the air out. The flexible ducts showed few, if any, of these trapped air-pocket voids. Although it is possible that the air in the pockets dissipated through the interlocking seams of the duct, a more likely explanation is that the corrugations present in the flexible duct walls provided a path for escape of the air.

Internal voids were generally quite small, of the order of ⅛ in. in diameter or less. They did not appear to collect at one point. Many of them were distributed along the wires. Their appearance was similar to the air bubbles exhibited in the plexiglas lab model. Some of the grout samples were found to be virtually free of voids. In most cases these samples were located at vent points in the interior of the ducts. The first 60 ft of the duct 321-A in which pressure leakage at the far end was prevented by the flash set were almost completely free of internal voids.

TABLE G-3

SAMPLE LOCATIONS AND OBSERVED PREVALENCE OF VOIDS IN FIELD GROUTING TESTS

			SAMPLE LOCATION			VOID PREVALENCE[d]		
DUCT SYSTEM	DUCTS IN SYSTEM[a]	GROUT TYPE[b]	DIST. FROM DUCT INLET (ft.)	DIST FROM DUCT OUTLET (ft.)	SAMPLE LOCATION ON DRAPE[c]	WATER TRACK VOIDS	EXTERIOR VOIDS	INTERNAL VOIDS
107-A	8R	A	22	85	low point	None	L	M
			43	64	positive slope	None	None	S
			64	43	high point	None	S	None
107-B	7F	B	22	85	low point	M	None	--
			43	64	positive slope	S	None	None
			64	43	high point	None	None	None
321-A	4F	A	22	299	neg slope	M	None	S
			43	278	low point	S	None	S
			64	257	pos slope	S	S	M
			85	236	high point	M	None	None
	5F	A	150	171	pos slope	M	M	None
			171	150	high point	L	None	--
			192	129	neg slope	S	S	None
	6R	A	236	85	high point	None	Shrinkage	None
			257	65	neg slope	None	Shrinkage	None
			278	43	low point	S	None	S
			320	0	outflow	No grout	No grout	--
321-B	1F	B	22	299	neg slope	L	S	M
			43	278	low point	M	M	--
			64	257	pos. slope	S	S	S
			85	236	high point	None	None	S
	2R	B	150	171	pos slope	S	S	None
			171	150	high point	None	S	M
			192	129	neg slope	M	M	--
	3F	B	236	85	high point	None	None	S
			257	65	neg slope	None	M	--
			278	43	low point	M	M	--
			320	0	outflow	None	Shrinkage	M

[a] Letter R following duct number signifies rigid duct, letter F, flexible duct

[b] Grout A, Type II cement with w/c ratio of 4.75 gal/sack, no additive
Grout B, same as grout A except with 0.5% Intrusion Aid by weight of cement

[c] In each case high point is at a vent, low point at a drain. Positive slope means grout was flowing up-hill at sample position, negative slope, down-hill

[d] Letters in columns 2 and 3 indicate number of voids observed, L = large number, M = moderate number, and S = small number. In column 1, L = a large water-track void, M = a moderate water-track void, and S = a small water-track void

Some corrosive effects of the tap water that was used for the duct-flushing operation were noted during the examinations for voids. The two 107-ft ducts, which were allowed to remain ungrouted for 3 days after flushing, showed only a slight rusting of the steel. On the other hand, the 321-ft ducts, which were allowed to remain ungrouted for 6 days after flushing, showed a very noticeable corrosion, especially where the wires were in contact with each other. There were also corrosion spots that indicated that water droplets had remained on the wires after the ducts were blown out with compressed air.

With the exception of the last 40 ft of duct 321-A, all of the examinations that were made after grouting showed that the ducts were well filled with grout and that no large voids were present. These findings are in general agreement with those of a recent investigation conducted for the Nuclear Electric Generating Station group of the Jersey Central Power and Light Co. (*64*). Both 0.5-in. strand and 0.25-in. wires were employed in that study. Two steel-to-duct area ratios (0.18 and 0.40) were used with the 0.25-in. wire tendons. Tendon lengths varied from 27 to 75 ft. Ducts were removed after grouting and examined for the presence of voids. Examinations apparently were made at saw cuts only (i.e., the metal wall of the duct was not removed, which would have permitted a longer section of the duct to be examined). The findings of the testing program were that normal grouting procedures gave effective grout penetration for all systems investigated and that no large voids

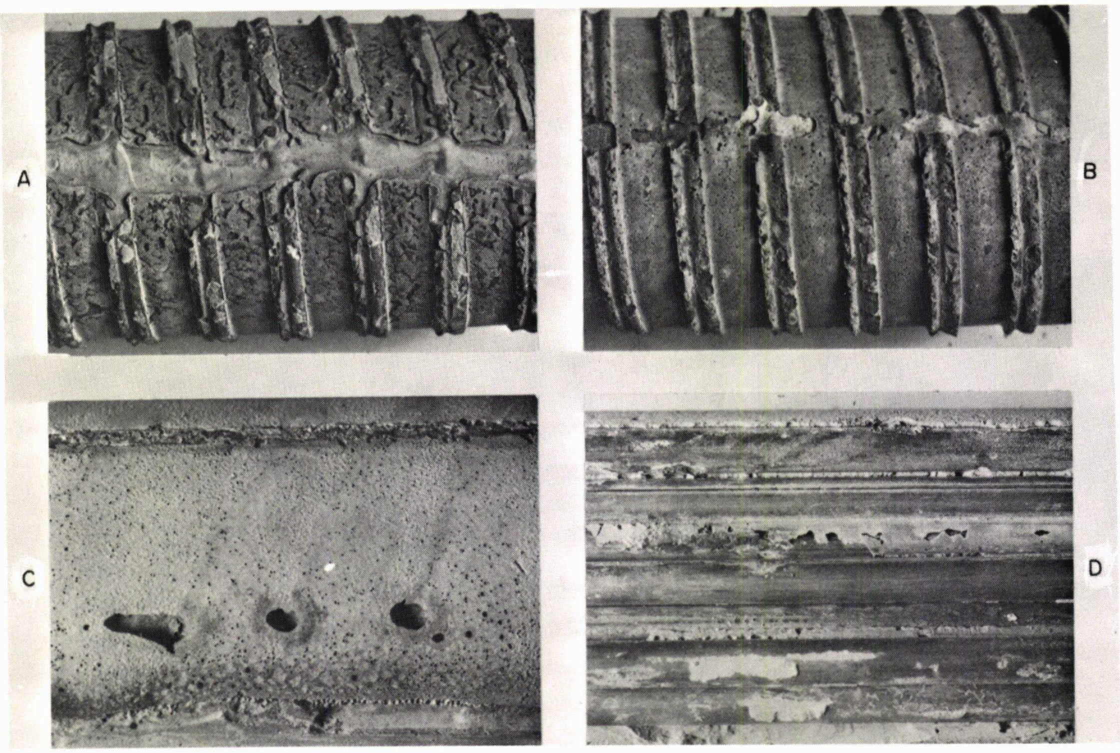

Figure G-18. Typical voids observed in field tests. (a) Water-track voids, severe; (b) Water-track voids, moderate; (c) Internal air voids; (d) Voids at steel-grout interface after removal of wire.

were formed. It was observed, however, that fewer small voids were generated at the steel surfaces for tendons formed with strand than for tendons formed with wires. The reason for this might be associated with the larger number of escape paths for entrapped air when seven-wire strand is in contact with the duct wall. Fewer escape paths of this type are present when smooth-surfaced wire tendons are used.

CONCLUDING REMARKS ON GROUTING STUDIES

Because of the finding that corrosion of prestressing steel can occur when sizeable voids are present in a simulated grouted duct, much of the effort in the grouting studies was directed to a better understanding of what needs to be done to minimize void formation in field grouting operations.

One important conclusion from the studies is that voids can be reduced in both size and number by maintaining the pumping pressure in the duct until the grout hardens. Unfortunately, maintenance of pressure during the initial setting period is not easily accomplished in field grouting operations. Sealing off of most practical systems is not always an effective method, because even small leaks will cause the pressure within the duct to dissipate within a few minutes. Grout pumps cannot be left operating to maintain the pressure because once setting of the grout begins to occur, the consistency of the slurry becomes too viscous for the pump to handle. Air pressure applied to a reservoir of grout after the pumping is completed should certainly be considered as one means of maintaining the pressure. The results of the work with plexiglas models suggests that there would be no appreciable penetration of air bubbles into the grout if this method was adopted.

The most serious types of voids observed during the course of the studies were those caused by water tracking. Shrinkage tests performed in the laboratory suggested that pressurizing the duct until hardening occurred would eliminate voiding of this type. The expansion of the grout caused by the presence of an additive was shown to be ineffective in eliminating the water-track voids.

The effect of an additive in reducing the size and number of voids caused by entrapped air (internal and external voids in Table G-3) was not too well defined in the field grouting tests. However, it is important to note that the expansion caused by the additive did not entirely eliminate voids of this type. Although only one additive was used, there is no reason to suspect that other expansive additives would behave differently, because all of them function by gas evolution during the initial stages of setting. The most important contribution of additives observed in the present work is their effect in increasing the fluidity of the grout. Although additives significantly lower the strength of the hardened grout, the strengths are still believed to be sufficiently high to permit the grout to perform its function of transmitting lateral and shear stresses from the prestressing steel to the concrete.

The plexiglas model tests indicate that internal voids are minimized by reducing the flow velocity of the grout. This can be accomplished by maintaining a partial backpressure on the duct during the pumping operation. One procedure might be to open only one grout vent at a time, thereby controlling the outflow pressures. However, it is also important to avoid the use of excessive pressures that can cause a "flash set" of the grout.

Control of grout fluidity is essential if void-free grout is to be achieved. The flow cone measurements provide information on the initial fluidity but cannot define the exact grout mixture. If a predetermined grout mixture has been defined to achieve a certain pumping distance without exceeding a particular pressure, flow cone measurements can be used to determine if the proper fluidity has been achieved. However, flow cone measurements cannot provide information as to the manner in which the fluidity was obtained. For example, the operator could simply add additional water and thereby bring the flow time to the required level. This would cause excessive shrinkage. If a suitable viscometer was used for field testing rather than a flow cone, such undesirable practices could be detected.

Only a single steel-to-duct area ratio (0.25) was used in the field grouting tests. From the standpoint of ease of pumping at reasonable pressures and of void formation by entrapped air, low ratios are the more desirable, especially for long girders. Because a "flash set" could conceivably occur at a pumping pressure above 150 psig, the safest procedure would be not to exceed an A_S/A_D ratio of 0.30 when a grout similar to grout B is to be pumped a distance of 300 to 400 ft. It should be pointed out that similar calculations are possible for grouts with other consistencies once the stress-strain characteristics of the selected grout have been determined.

Two types of ducts (flexible and rigid) were incorporated into the field testing. The flexible type showed the presence of fewer air-entrapped voids at the duct wall. This observation—together with the greater ease of draping flexible ducts in the forms prior to casting—suggests that flexible ducts might be superior to rigid ones. On the other hand, rigid ducts provide a better seal against the later entrance of corrosive solutions, and also there is somewhat less likelihood with rigid ducts of cement paste penetrating the duct wall to cause an obstruction during later grouting operations. No significant difference in pumping characteristics was noted in the field testing between the rigid and flexible conduits.

One observation of importance in the field grouting tests was the significant amount of corrosion that occurred while the steel was in the ducts prior to grouting. Laboratory testing indicated that testing the ducts for obstructions with saturated $Ca(OH)_2$ solution rather than water would have largely eliminated this corrosion. Possible methods of protecting against corrosion when ungrouted steel is to be left in ducts for long periods are discussed in Appendix E.

Present Recommended Practices for Grouting Post-Tensioning Ducts

Two tentative recommended practices for grouting post-tensioning tendons are in current use in the United States. The first (*97*), issued in 1960 by the Prestressed Concrete Institute, is entitled "Tentative Recommended Practice for Grouting Post-Tensioned Prestressed Concrete." The second (*98*), prepared jointly by the Prestressed Concrete Manufacturers Association of California and the Western Concrete Reinforcing Steel Institute, is entitled "Recommended Practice for Grouting Post-Tensioning Tendons, July 1967 (Tentative)"; it contains more detailed recommendations on grouts and grouting practices than the one issued by PCI. Hence, the comments that follow apply to the PCMAC recommended practices.

Section 2.1 of the PCMAC "Recommended Practice" suggests that the grouted tendon must be completely filled, with no voids occurring either from entrapped air or entrapped water. The field tests clearly indicated that the elimination of small voids (regardless of their size and distribution) is a most difficult task. The test results showed that to achieve a condition even approaching zero voids, a *retained* pressure seal must be attainable for the ducts. Present procedures for fabricating tension plates for the tendon wires with a button-head design do not permit such a seal to be obtained, and leak-tight installations may also be difficult to achieve with other systems as well. Because inspection for the presence of small voids in a grouted duct is virtually impossible, it is recommended that additional work be done on possible methods of retaining pressure in ducts while the grout is hardening so as to reduce void prevalence. Also, it is recommended the wording of Section 2.1 be changed from "with no voids" to "with no voids greater than ⅛-in. diameter."

Section 4.5 restricts the water-cement ratio to not greater than 5.0 gal/sack. Grouts with ratios higher than 5.0 have quite high shrinkages and low strengths after curing. This is especially true when an admixture is used with grouts having w/c ratios higher than 5.0. Thus, the limit of 5.0 would seem to be a realistic limit, as well as a practical one for field applications.

Section 4.6 states that an additive shall be used and that it shall provide a 5 to 10% unrestrained expansion of the grout. Although the test results indicated a definite increase in the flowability of the grout with the use of an additive, there was little evidence that it provided any benefits to the grout-curing process or that it contributed in a very important way to the elimination of voids. It would seem appropriate, therefore, to describe grouts in terms of their flow properties, strength, and shrinkage characteristics rather than whether they contain an additive.

Section 5.3 suggests that a pressure gauge be located somewhere between the pump outlet and the duct inlet. The length of the grouting hose and its pressure drop can be significant in defining the location of the gauge. A 100-ft length of 1-in. rubber grouting hose can exhibit a pressure drop of as much as 50 to 60 psi. Thus, if the maximum grouting pressure is to be limited to 150 psig, the gauge should be located at the pump.

Section 6.3 states that a trial batch shall be tested by the Engineer using the standard flow cone method. As stated earlier, the flow cone is not a completely suitable method for characterizing the flow properties of grout. However, more work is necessary before a more meaningful type of measurement can be recommended for field testing. In the meantime, use of the flow cone test should be continued.

Section 6.4 states that the trial batch should exhibit an efflux time of at least 11 sec. This value is typical of a

5.0 w/c ratio mix with a small percentage of additive. Unless additional information on the characteristics of the mixture is needed, this test value appears to be an adequate threshold. An additional test, suggested in Section 6.4, is that the efflux time of the grout be measured after it has been allowed to stand for 20 min. The value of this test, however, is questionable. Grout mixes tend to segregate when allowed to stand for more than 5 min without agitation. Thus, a 20-min test would not give a true measure of the fluidity of the grout in the container. It is recommended that additional work be done to evaluate the potential usefulness of a properly designed commercial viscometer. Because a viscometer of this type would measure shearing stress for varying strain rates, its potential for defining pumping pressures and pumping distances for varying field operations is much greater than the flow cone.

Section 7.1 states that duct flushing is recommended, and that limewater should be used for the flushing operation. The study concurs with the need for limewater to minimize corrosion. However, the need to flush a metal-lined duct with water is questionable. An air pressure test should indicate if any major blockages exist, or if there is a major leak. It is doubtful that all foreign matter within the duct is removed by the flushing. On the contrary, some of this material could be lodged under the stressed wires by the flushing operations and thereby form additional voids.

Section 7.3 states that the initial pumping pressure shall be less than 40 psig and increased only as needed to keep the grout flowing. The experience from the present field tests indicates that the grout velocity can be better controlled by keeping the outflow valves partially closed to create a small backpressure. Definition of standard procedures should be based on additional experience with this approach because of its tendency to magnify the problems associated with excessive pressure. No change in the suggested maximum pressure of 150 psig is recommended.

Section 7.8 states that the grouted duct should be closed with a retained pressure of at least 60 psig. It is recommended that this pressure be maintained until the grout reaches an initial set. Because of the occasional leakage in practical duct systems, this recommendation could require that an air-pressurized grout reservoir be attached to the ducts for a period of several hours after the grouting is completed.

APPENDIX H

EXAMINATION OF PRESTRESSED MEMBERS AFTER LONG-TIME EXPOSURE

Much could be learned about the corrosion behavior of steel in prestressed concrete if it were possible to remove and examine the steel from bridges that had been in service for a number of years. Because, for obvious reasons, removal of steel would never be permitted for in-service bridges, it was necessary instead to search for structures that were out-of-service because of some such reason as flood damage, relocation of a highway, or a serious traffic accident. In fact, one of the purposes of the highway department survey described in Appendix A was to uncover out-of-service bridge structures. However, only three were reported, and fragments of only one of these three were available for examination.

Inspection of steel in other types of prestressed structures after long-time exposure could also be enlightening on the type of steel corrosion that might be expected in bridges. Results of an examination of one such structure, a prestressed pile, is included in this appendix.

EXAMINATION OF A GIRDER FRAGMENT FROM AN OUT-OF-SERVICE BRIDGE

A fragment of a pretensioned girder from an out-of-service highway bridge near Ft. Morgan, Colorado, was examined for steel corrosion. The bridge had been destroyed by a severe flood in 1965 after approximately 2 years of service. The fragment, after the final bridge demolition, rested on the bank of the South Platte River for 1.5 years. The concrete cover over the strands was 2 in.

Figure H-1 shows a section of one of the 0.5-in.-diameter strands embedded in the concrete matrix. Note that the interstices between the center and outer wires are filled mostly with hardened cement paste. This paste presumably permeated through the very small separations between the outer wires of the strand during placement of the concrete.

The girder fragment had short lengths of strand projecting from the concrete. This permitted easy access of moisture at the ends. Corrosion was found to have progressed inward along the center wire for a distance of about 2.5 in. from the point where the strand entered the sound concrete. This indicated that the hardened cement, which filled approximately 50% of the strand interstices, had retarded moisture penetration. If the paste had not been present, a much deeper penetration would have occurred.

The laboratory examination showed that no corrosion had occurred on any of the outer surfaces of the strand that had been in contact with concrete.

EXAMINATION OF STEEL REMOVED FROM A PRESTRESSED PILE

Two sections of ⅜-in.-diameter strand were examined after 11 years of exposure in a marine environment. The pile, of octagonal cross section and 14 in. between opposite faces, was manufactured in 1956 (by the Prestressed Div., Ben C. Gerwick Co., Petaluma, Calif.). The pile proved to be too long when driven and it was necessary to remove the top 10 ft with pneumatic tools. The section cut from the pile was then placed on the bank of the Petaluma River under a parking wharf (Fig. H-2) where for the next 11 years it was subject (because of the tide) to a twice-daily immersion in brackish salt water. The Petaluma River is brackish only when rains occur, which is usually in the spring; the remainder of the time the salinity of the water approaches that of San Francisco Bay.

Records of the manufacturer provided the following information:

1. Date of manufacture—1956.
2. Date of installation—Fall 1956.
3. Cement type—ASTM Type II.
4. Cement content—7.5 sacks per cubic yard.
5. Water content—4.5 gal per sack.
6. Slump—2.5 in.
7. Curing—steam cured overnight at max. temp. of 160°F.
8. Strand diameter—⅜ in.
9. Number of strands—11.
10. Strand spec (UTS)—250 ksi.
11. Steel surface condition—some surface rusting prior to encasement in concrete.
12. Concrete cover—2 in.

The strand was removed from the pile section in January 1968.* Two 3-ft-long sections of the strand were then forwarded to the researchers' current laboratory for examination.

Figure H-3 shows the pile section after the concrete had been chipped away to expose the strand. The following summarizes the findings for the part of the strand that was encased in concrete:

1. Some rust spotting was observed on wire surfaces. The spots were somewhat more prevalent on the surfaces of the six outer wires of the strand than on the center wire. According to the manufacturer, the practice in 1956 was to store the prestressing steel out-of-doors, and it is probable that the observed rust was already on the strand at the time the pile was fabricated.
2. Very little pitting of the steel had occurred from the rusting. The maximum pit depth was 1.4 mils. This was observed at one location on the center wire.
3. Corrosion had not progressed inward from the broken end of the pile for any appreciable distance (less than 3 in.) except at the extreme top end where the concrete had become cracked during the pile-driving operation (top 18 in.).
4. The interstices surrounding the center wire were estimated to be 75% filled with hardened cement paste.

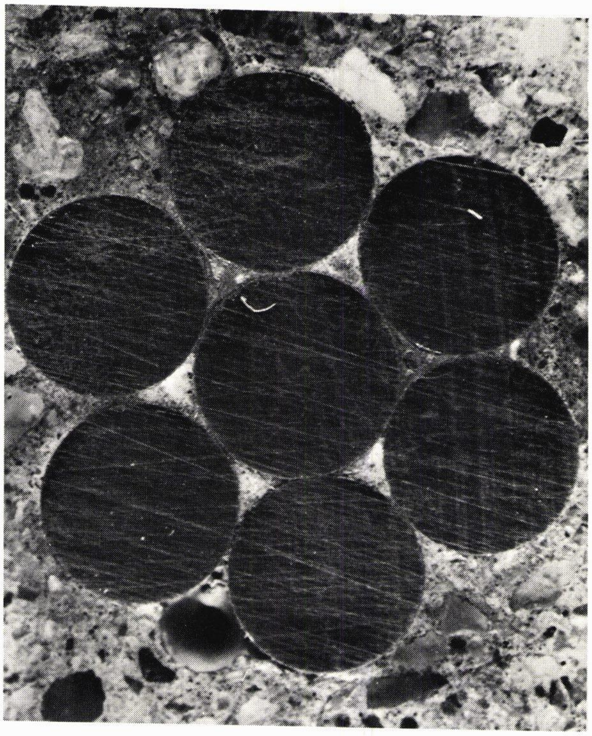

Figure H-1. Section of 0.5-in. strand in pretensioned girder fragment after 1½ years on bank of South Platte River, Ft. Morgan, Colo. Note cement paste in strand interstices.

Figure H-2. Exposure site of 10-ft pretensioned pile section on Petaluma River bank, California.

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* At the direction of Ben C. Gerwick, Jr., President of the Ben C. Gerwick Div. of J. H. Pomeroy & Co., Inc., San Francisco.

Figure H-3. Appearance of steel strand after chipping away concrete cover.

5. One 20-in.-long section of the center wire of the strand was tensile tested. Failure occurred in the grip at 238,000 psi. The steel had just entered the plastic range at the time of grip failure. Therefore, because this plastic range is usually observed at about 85% of the failure stress, it would appear that the ultimate tensile strength of the wire sample after the 11-year exposure period was well over 250,000 psi and was probably above 270,000 psi.

6. A 5-ft-long piece of strand, taken from the center of the 10-ft-long section, was tensile tested by the Sonoma Testing Laboratory, Santa Rosa, California. The values obtained after the 11-year exposure were: 0.2% offset yield—250,800 psi; ultimate tensile strength—281,800 psi.

7. One longitudinal crack was observed in one of the outer wires when the strand was sectioned. Because similar cracks were observed occasionally on wires for ⅜-in. strand that were sent to the researchers' laboratory by a steel mill, it is at least possible that the observed longitudinal crack was present in the wire at the time of fabrication.

SUMMARY

Both of the examinations confirmed that concrete is capable of providing excellent protection for pretensioning steel, even under very severe environments. The girder fragment, which rested on a riverbank for 1½ years after being removed from a bridge that had been in service for 2 years, showed no evidence of corrosion of the steel strand, except for a mild corrosion of the center wire near the ends.

The salt-water exposure of the pile fragment was much more severe than any conceivable environments that could be envisioned for bridge girders. Yet, after 11 years of the twice-daily immersion in brackish salt water, the pretensioning steel still maintained its initial properties. Corrosion of the steel occurred only near the ends, and even at the ends penetration inward along the center wire was minimal because of the presence of hardened cement paste in the interstices of the strand. This paste, which penetrated the strand during vibration placement of the concrete, appeared to be quite effective in blocking the capillary flow of liquids along the center wire. The paste was found to be present in the strand of all pretensioned beams that were examined during the course of the study.

APPENDIX I

REVIEW OF POSSIBLE METHODS FOR INSPECTING IN-SERVICE BRIDGES

VISUAL INSPECTION

The most common method for detecting steel corrosion in reinforced concrete structures is to inspect the structure for rust staining. Rust stains that follow the line of the reinforcing steel are always good evidence of steel corrosion. When the corrosion becomes more severe, cracking of the concrete over the steel often occurs. The cracking is caused by the pressures generated by the corrosion products on the steel, because these products always occupy a larger volume than the uncorroded steel plus the reactants. The corrosion products have been shown to occupy 2.2 times as much space as the metal. Mechanical pressures up to 4,700 psi can be generated by their presence (*99*).

The visual inspection method is, of course, used on prestressed structures. It may not, however, always be reliable because, with prestressing wires, only a small amount of corrosion at a localized area could be dangerous and this could occur with no visual evidence on the outer surface of the concrete. Conversely, rust staining over stirrups, which normally have less concrete cover than the prestressing steel, would not be an indication that the prestressing steel was corroding. At the same time, it would be an excellent indication that corrosive conditions were present in the girder.

POTENTIAL MAPPING

Flor and Keith (*36*) made potential surveys of three large prestressed concrete tanks used for water storage. All three tanks were in contact with the soil; hence, a reference electrode could be set in the soil near the tank. Potential reading were taken with a copper-copper sulfate roving electrode at points on a grid marked off on the tank wall. The electrode was tipped with a cellulose sponge saturated with copper sulfate solution. The outer walls of the tanks were saturated with water prior to obtaining the measurements. The investigation showed that areas of steel corrosion could be located by this approach. In one tank, grooves were chipped in the concrete at eight areas of high potential difference so as to expose the prestressing steel. Corrosion was not always the most severe at the highest potential points, but the area of most severe corrosion was always within 2.5 ft of the highest potential.

Tremper, Beaton, and Stratfull (*4*) used copper sulfate half-cells to measure potentials on the reinforced concrete beams of the San Mateo-Hayward Bridge over San Francisco Bay. The measurements were made on a grid pattern over the exposed surfaces of the beams. On completion of the measurements, a series of curves or contours was plotted through the points of equal potential. These equipotential contours, if of positive polarity, were found to close around anodic areas. Later inspections showed that the embedded steel was corroded in these areas.

In way of summary, it appears that whereas potential mapping is a promising approach to locating areas of localized corrosion of prestressing steel in a prestressed bridge structure, further work will be required before the method can be used with confidence by anyone except a highly experienced corrosion engineer.

CORROSION PROBES

Griffin (*21*) designed a corrosion-detection probe to study the corrosion of steel in concrete under various corrosive environments. Even though Griffin did not develop the device to detect the onset and the rate of corrosion in reinforced concrete structures, it would appear that it might be useful for this purpose.

The probe consisted of a hairpin of steel attached to a terminal head. The probe was cast into the concrete and the change in resistance due to loss of steel by corrosion was measured periodically. To improve precision, the measurement was made using a bridge circuit in which a second probe, protected from corrosion, was used as a comparison standard.

One serious objection to embedding probes of this type in a girder during manufacture, so as to function later as corrosion detectors, is that they could not reproduce the long-range corrosion couples such as those observed by Tremper, Beaton, and Stratfull (*4*). Unless this feature could be incorporated, the value of such probes would be open to question.

RESISTANCE MEASUREMENTS OF CONCRETE

Measurements of the resistance of the corrosive environment have not been employed widely in corrosion testing. On the other hand, there are indications that this type of measurement could be employed much more extensively, particularly for those types of corrosion that are known to be resistance controlled (*100*). Concrete (like soils and some ground waters) normally has a fairly high resistance. Thus, if low resistances were measured, it might be an indication that a corrosive environment existed within the concrete.

That such is probably the case is indicated by the measurements of Tremper, Beaton, and Stratfull (*4*). Figure I-1 (taken from their data for San Mateo-Hayward Bridge spans) shows the relationship that was observed between electrical resistance and concrete deterioration. The concrete deterioration was taken as the relative length of longitudinal cracks along the bottom of the beams (the cracks were presumed to be caused by the pressures generated by the formation of corrosion products on the reinforcing bars). Details regarding the resistivity measurements were not included in the report.

At least for this particular installation, in which sea salts

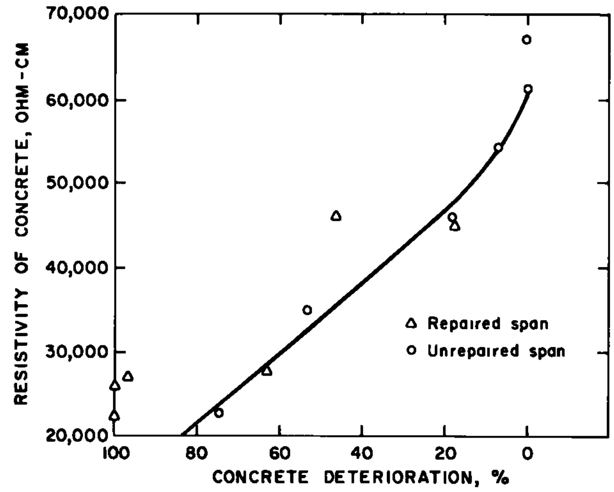

Figure I-1. Relation between resistivity of concrete in anodic areas of spans and concrete deterioration caused by reinforcing bar corrosion as measured by relative length of cracks along bottom of girders (4).

were present, the degree of corrosion was related to the resistivity of the concrete. When the resistivity was above 60,000 ohm-em, no corrosion occurred; when it was below 60,000 ohm-em, corrosion was detected—the lower the resistivity the greater the corrosion of the steel.

These observations at least suggest the possibility of monitoring prestressed bridge spans by means of resistance measurements to determine if corrosive conditions exist within the concrete. Because water content affects the resistivity, such measurements would need to be made while the spans were wet. Also, they would need to be made on a grid pattern to be certain the local anodic areas were being monitored. The resistivity of anodic areas was found by Tremper *et al.* (*4*) to be always lower than the cathodic regions.

It should be strongly emphasized that this resistance approach would need considerably more study before it could be recommended as a method for determining when serious corrosive conditions existed in a bridge structure.

RECOMMENDED PROCEDURE FOR INSPECTING PRESTRESSED BRIDGES FOR STEEL CORROSION

Periodic visual inspections of the structural members should be made on all in-service bridges. The frequency of these inspections will be related to the severity of the environment with respect to corrosion. For bridges over sea or brackish water, or in highly polluted industrial atmospheres, semi-annual inspections might be desirable; for other bridges, annual inspections should be sufficient.

When making those inspections, inspectors should note in the report the presence or absence of the following evidences of steel corrosion:

1. Rust staining of concrete at any location along tendons.
2. Longitudinal cracks in girder.
3. Transverse cracks in girder.
4. Deterioration of concrete by spalling or from other causes.

Rust staining that appears at the concrete surface of the girder is an almost sure indication that steel corrosion is occurring. When and if rust staining appears, more frequent inspections are indicated, with a record being kept of the growth in size of the stained areas.

Longitudinal cracks along a concrete girder (either with or without an accompanying rust staining) are a very strong indication that serious corrosion of the steel has occurred. The appearance of transverse cracks—especially at points of maximum stress—suggests that corrosion of tendons has progressed to a point where the steel is no longer capable of carrying the applied load. Failure may be imminent when open transverse cracks appear in any girder that has been in service for an extended period.

Hairline-type shrinkage cracks and small shear cracks that appear at the supports, although undesirable, are normally not signs of steel corrosion, nor are those cracks that are caused by reactive aggregates or by freezing and thawing. Likewise, cracks over post-tensioning ducts caused by freezing of grout before it is completely cured cannot be ascribed to corrosion.

Particular attention should be given during the inspections to any changes in the quality of the concrete over the tendons. Spalling is especially undesirable because it reduces the effective cover for the steel. Loss of soundness by attack of acid gases in the atmosphere will usually be evidenced by a soft punky structure in the concrete.

No method exists for inspecting tendons for possible incipient failure due to brittle fracture. Fortunately, however, this study shows that prestressing steel when embedded in concrete is not susceptible to failures of this type (see Appendix C).

APPENDIX J

CONCLUSIONS AND RECOMMENDATIONS FROM CURRENT PRACTICES REPORT *

Considering the large number of prestressed concrete highway bridges and the limited amount of damage due to corrosion of prestressing steel, it can be concluded that high-quality impervious concrete and grout can furnish excellent corrosion protection of the prestressing steel, provided that certain requirements are satisfied.

Corrosion problems in prestressed concrete bridges are caused mostly by improper construction practices and to a lesser degree by improper design. Although the current AASHO *Specifications* (*52*) are inadequate concerning requirements on corrosion protection of prestressing steel, the present practice in the U.S. can be considered "generally safe," largely due to the efforts of the prestressing and cement industry, which tries to safeguard against all possible failures by developing tentative specifications covering all phases of construction of prestressed concrete members (*97, 102, 103*).

At present, more than 90% of all prestressed concrete bridges are pretensioned. This percentage is considered to be somewhat misleading, because pretensioning is used mostly for small-span bridges. A more realistic percentage can be obtained by comparing dollar values, which yields an estimate of 75%. It is believed that post-tensioning will play a more important role in the future construction of prestressed concrete bridges, because it is equally suited for mass production of small-to-medium-long-span bridges and for *in situ* application in case of medium- and long-span bridge girders. The survey has shown that the Freyssinet and Stressteel systems are the most widely used post-tensioning systems in the United States.

Based on the survey, the researcher considers the current pretensioning practice somewhat safer with regard to corrosion protection of prestressing steel than post-tensioning, because the concrete (which provides corrosion protection in case of pretensioning) is of exceptionally high quality; furthermore, the high quality is easier to insure than in cases of post-tensioning, which usually takes place at the job site. Only minor improvements are recommended in certain pretensioning operations, whereas, in some cases, the grouting techniques leave much to be desired. The steel used in pretensioning and in post-tensioning in the U.S. is of cold-drawn, stress-relieved type and is considered to be less susceptible to stress corrosion than "heat treated" (tempered) wires sometimes used in other countries.

Definite improvements are required for shipping, storing, and handling of prestressing steel. It can be stated that the manufacturers of prestressing steel generally do not realize the importance of proper packing and the use of corrosion inhibitors during transportation. A closer cooperation between the manufacturer and the prestressing industry is recommended to make the manufacturers aware of the actual needs of the users. In the majority of the cases, the storing of prestressing steel is inadequate. It has been found that dragging of prestressing steel on the ground, which can be chemically aggressive, is not uncommon. The time between tensioning the steel and applying the protective concrete or grout should be considerably reduced.

The other area which requires improvements is the presently used grouting techniques. A definite need exists to correlate the viscosity of the grout with its void-filling characteristics and with the optimum duct-to-tendon area ratio. Mixing of grout by concrete mixers and even placing the grout by compressed air has been reported. Until a revised AASHO code is available, the use of the International Recommendations (see Appendix L) jointly with the "Tentative Recommendations" (*97*) of the Prestressed Concrete Institute is strongly recommended. On the other hand, it has been found that the quality of grout used is high. The corrosion protection of the anchorage either by coating or by mortar is considered mandatory. The researcher does not recommend the use of unbonded prestressed girders for bridges, considering various safety and economic factors.

The effectiveness of the presently used concrete covers (*97*) to provide corrosion protection of the prestressing steel appears to be generally adequate.

Because corrosion generally is a long-time process, and because the detection of corrosion is extremely difficult, the conclusion that the present relatively corrosion-free conditions will also exist in a more distant future would be misleading. Consequently, revised AASHO specifications, which cover explicitly the items described and recommended in the report, are required. There is also a pressing need for establishing proper field tests and inspections which will ensure the required anti-corrosive properties of the concrete and grout.

Finally, in view of the possibility of more extensive latent corrosion problems, periodic corrosion inspection of prestressed concrete bridges—along with development of more effective devices for detection of corrosion—are highly recommended.

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* Professor R. Szilard served as one of the principal contributors on this project during its first year. Because of his long association with design and construction of prestressed concrete structures, both in the United States and Europe, he was especially well qualified to survey and evaluate prestressing practices with respect to corrosion protection.

Professor Szilard completed this assignment prior to his resignation from the University of Denver in the summer of 1967. The survey was conducted by means of questionnaires, personal correspondence, and personal visits. A comprehensive interim report was prepared and submitted to the Highway Research Board in May 1967 (*101*). Because of the length of the report (63 pp.), only Professor Szilard's conclusions and recommendations are included in this appendix.

APPENDIX K

RECOMMENDED CHANGES IN AASHO BRIDGE SPECIFICATIONS

The *Standard Specifications for Highway Bridges* as adopted by the American Association of State Highway Officials (*52*) contains 12 pages on requirements for prestressed concrete. The only part of these requirements that applies either directly or indirectly to corrosion protection of tendons is one brief section on minimum cover (*Section 1.6.16A*).

As a result of the findings of the present investigation, including those of the literature survey, it is recommended that *Section 6* on Prestressed Concrete be expanded so as to provide an improved insurance against possible future corrosion damage to tendons. The additional requirements that should be incorporated are listed as follows:

Prestressing Steel: The steel shall meet the requirements of ASTM Specification A-416 for seven-wire, stress-relieved strand; A-421 for uncoated stress-relieved wire; or A-311 for stress-relieved, cold-drawn carbon steel bars; whichever is applicable. The steel shall be free of such defects as longitudinal cracks, chevron cracks, and surface notches. No welding shall be permitted. The tendons at the time of use shall be free of rust that cannot be removed by wiping with a dry cotton cloth. Dragging of tendons on corrosive soils shall not be permitted.

Concrete: The concrete in prestressed bridge members shall be prepared with portland cement meeting the requirements of either ASTM Standard Specification C150 or ASTM C175. The aggregate shall be nonpermeable and of proven resistance to alkali attack (alkali-aggregate reaction). No calcium chloride shall be used as an admixture. The maximum weight percentage of soluble chloride in the ingredients used for preparing the concrete shall be as follows:

Portland cement	0.10%
Mixing water	0.03
Sand	0.02
Aggregates	0.02

The concrete shall be prepared with not less than 6.0 bags of cement per cubic yard of concrete and the water-cement ratio shall be not greater than 5.0 gal of water per bag of cement.

Lightweight concrete shall be permitted for prestressed bridge girders if its use is mutually agreed on by the contracting parties.

Minimum Concrete Cover: For pretensioned members the following minimum concrete covers shall be provided:

	Normal Locations	Marine or Industrially Polluted Sites
Prestressing steel and main reinforcement	1.5 in.	2.0 in.
Slab reinforcement:		
Top of slab	1.5	2.0
Bottom of slab	1.0	1.5
Stirrups and ties	1.0	1.5

For post-tensioned members the minimum cover between the duct walls and the surface of the concrete shall be 1 in., except that for bridges constructed in marine or industrially polluted atmospheres the minimum cover shall be 1.5 in.

Grouping of Post-Tensioned Tendons: The tendons of post-tensioned girders shall be grouted with portland cement grout. The grout shall have a water/cement ratio of not greater than 5.0 gal/bag of cement. The use of any admixture that contains a chloride shall not be permitted. Pumping of the grout shall be done in such manner as to assure complete filling of the duct.

APPENDIX L

INTERNATIONAL RECOMMENDATIONS FOR GROUT AND GROUTING OF PRESTRESSED CONCRETE

A set of recommendations for grouts and grouting of tendons as prepared by a joint FIP-RILEM Committee on Grouting was incorporated as an appendix to the interim report by Professor R. Szilard on current prestressing practices (*101*). Because this FIP-RILEM report is not readily available in the United States, the recommendations of the committee are repeated in this appendix.

1. Grouting the ducts of post-tensioned concrete members has two main objectives:
   (a) to prevent corrosion of the prestressing steel;
   (b) to provide an efficient bond between the prestressing steel and the concrete member.
2. Properties of the grout:
   (a) The water/cement ratio should be as low as possible, consistent with adequate workability. It is recommended that the fluidity of the grout should be measured on the site as a method of control. There are several types of apparatus which give reproducible results and if these are calibrated in poises the results can be interpreted on an international basis.
   (b) The bleeding of the grout at 18°C should not exceed the following: 2% of the volume 3 hr after mixing; a maximum of 4%. In addition, the separated water must be absorbed after 24 hr. Bleeding should be measured in a metal or glass cylinder with an internal diameter of approximately 10 cm, with a height of grout of approximately 10 cm. During the test the container should be covered to prevent evaporation.
   (c) The compressive strength of the hardened grout after 28 days at a temperature of 18°C and a relative humidity of approximately 70% should not be less than 300 kg/cm^2 for small cubes (7- or 10-cm side) or cylinders of approximately equal height and diameter. If cylinders of other relative dimensions are used a conversion factor may be applied.
3. Materials for grout:
   (a) Cement should be portland cement or portland blast furnace cement of medium fineness. The cement must not contain calcium chloride.
   (b) If aggregates are used they should consist of ground quartz or limestone, trass, or fine sand.
   (c) Additives may be used if tests have shown that their use improves the properties of the grout; e.g., by increasing workability, reducing bleeding, entraining air, or expanding the grout. Additives must not contain chlorides or nitrates. When an expanding agent is used, the total unrestrained expansion should not exceed 10%.
4. Mixing grout:
   (a) The cement (and aggregates if used) for grout should be measured by weight.
   (b) The mixing equipment should be of a type capable of producing grout of uniform, and, if possible, colloidal, consistency.
   (c) Water should be added to the mixer first, then the cement and aggregate (if any). Additives, if used, should be added during the latter half of the mixing time.
   (d) Mixing time depends on the type of mixer, but will normally be between 2 and 4 min.
   (e) Mixing by hand is not recommended.
5. Injecting grout:
   (a) After being mixed, the grout should be kept in continuous movement. It is essential that the grout should be free of lumps.
   (b) It is preferable to flush the ducts with water before injecting the grout. Where ducts have concrete walls it is essential to ensure that the walls are thoroughly wetted before commencing the injection. After flushing, surplus water should be removed by compressed air or other means.
   (c) Injection must be continuous and should not be interrupted. The pump should be of sufficient capacity to ensure that in ducts less than 10 cm in diameter the velocity of the grout is between 6 and 12 m per minute with a pressure not exceeding 10 atmospheres.
   (d) The pump should be fitted with an effective control against buildup of excessive pressure.
   (e) The maximum allowable pressure will depend on the cross section of the duct and the thickness of concrete cover.
   (f) Injection by compressed air is not recommended.
   (g) Whenever possible, injection should be effected from the lowest anchorage or vent of the duct.
   (h) For very large ducts it may be necessary to inject a second time after about 2 hr.
   (i) Injection must be continued until the consistency of the grout flowing from the free-end vent openings is equal to that of the injected grout.
   (j) The connection between the nozzle of the injection pipe and the duct should be positive and such that air cannot be sucked in.

(k) Immediately after injection, measures should be taken to prevent loss of grout from the duct. The injection tubes should subsequently be topped up with grout if necessary.

Note: Disconnection is facilitated if a short length of flexible tube connects the duct and the injection pipe. This can be squeezed and cut off after the grout has hardened.

6. Ducts:
   (a) Sudden changes in alinement and section should be avoided as far as possible.
   (b) Vents may be provided at the crests of the ducts.
   (c) To prevent water collecting in the ducts before grouting, a vent should be provided in the valleys.
7. Grouting in cold weather:
   (a) In frosty weather, injection should be postponed, unless special precautions are taken.
   (b) If it is certain that the temperature of the structure will not fall below 5°C during the next 48 hr, grouting may continue, using a frostproof grout containing 6 to 10% of entrained air.
   (c) If frost is likely to occur within 48 hr, heat must be applied to the member, and maintained for at least 48 hr after injection so that the temperature of the grout does not fall below 5°C.
   (d) After a period of frosty weather, care must be taken to ensure that the ducts are completely free of frost and ice before commencing grouting (e.g., by flushing with warm water, but not with steam).

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NATIONAL ACADEMY OF SCIENCES—NATIONAL RESEARCH COUNCIL
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